Considerations in Economical Design of Jointed Plain Concrete Pavement

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.....*Santhosh*

CONSIDERATIONS IN ECONOMICAL DESIGN OF JOINTED PLAIN CONCRETE PAVEMENT

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Chapter 1

Introduction

1.1. General

Rapid economic development has led to an explosive growth of vehicle population in our country. The total length of National Highways in India is 58112 km which is about 2% of the total road length while it accounts for 40% of the vehicular traffic (*CP-2002, 2002*). The mounting stress on transportation infrastructure has forced policy makers to consider the construction of concrete highways which are long lasting and maintenance free. Improved and dependable road infrastructure is now being given high priority by the government. For modernizing and upgrading our road system to make it world class, it would be imperative to provide with long lasting, maintenance free pavements and concrete pavements meet the criteria. Concrete has been used in pavement construction in different parts of the world for the last several decades. The United States recognizing the benefits of a concrete infrastructure has incorporated concrete pavement into over 30% of its interstate highways *(FHWA, 1997)*.

Professional organizations in different countries have developed relevant design methodologies and construction methods to suit local conditions. Specifications published by American Association of State Highway and Transportation Officials (AASHTO), National Association of Road Transport and Traffic Authorities in Australasia (AUSTROADS), Design manual for roads and bridges, Transport and Road Research Laboratory (TRRL) UK, Portland Cement Association (*PCA*) etc., are examples of such

guidelines. Indian specifications on design of rigid pavements are given in IRC 58-2002. However, the history of such rigid pavements in India is relatively recent, and an extensive effort is being made only now during the construction of the national highways. Thus, the Indian documents dealing with various aspects of design and construction of concrete pavements need substantial changes in order to make them more relevant to local conditions, and ensure economy.

As a first step towards identifying the areas in Indian codes and specifications where improvements are needed, a study was carried out to better understand the relevant provisions in some of the international standards. This thesis presents the findings of the study, along with a comparative analysis of some of the international codal provisions and analysis of some innovative pavement design which may help in construction of economical concrete pavements. A basic study using the Structural Analysis Programme (SAP) was carried out to better understand the stresses developed in concrete pavements during operation. This analysis was also carried out using some innovative pavement geometries with a view to economize pavement construction.

1.2 Background: Rigid Vs Flexible pavement

There are two types of pavement constructions which are followed all over the world: the flexible which basically uses the bitumen as the binding material with the pavement being elastic, transferring most of the load applied to the lower layer and rigid pavement where the binding material is cement. Here the concrete's rigidity and stiffness, tends to distribute load over a relatively wide area of sub grade. The advantages and disadvantages of both the construction practices are discussed as under:

a) **Basic structure:** The concrete slab itself supplies a major portion of a rigid pavement's structural capacity. The rigid nature spreads the load over a large

area while in flexible the elastic nature tends to transfer most of the load to the under lying layers. This is depicted in Figure 1.1.

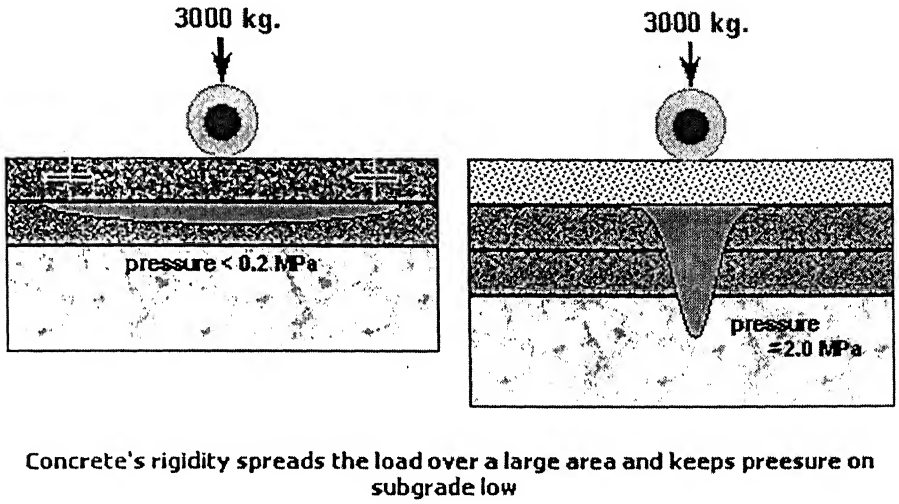


Figure 1.1: Distribution of load by concrete and bituminous pavements (*CAC, 2002*)

b) **Deformation on loading:** The rigidity of the concrete prevents the pavement prevents deformation from occurring and thus they do not rut, washboard or shove while in asphalt roadways heavy vehicles cause greater deflection on flexible pavements causing ruts, while the stopping and starting motion of a heavy vehicle create a wash boarded surface *(CAC, 2002)*. A study by the AASHTO Road Test showed that 61% of asphalt roads fail during spring conditions compared to 5.5% for concrete pavements *(CAC, 2002)*.

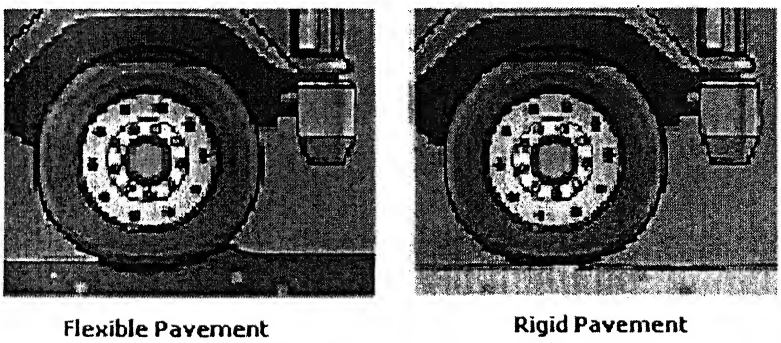
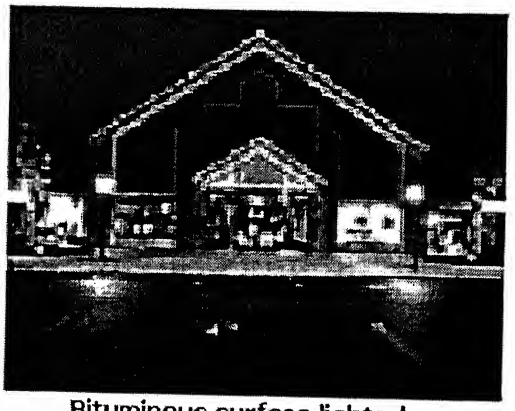


Figure 1.2: Deflection of flexible and rigid pavement due to vehicle loads (*CAC, 2002*)

c) **Fuel economy:** The flexible nature of the pavement causes it to deflect on heavy loads thus absorbing part of the vehicle energy that would otherwise be available to propel the vehicle as shown in Figure 1.2. This leads to an increase in the consumption of fuel. *(Report on Fuel Savings for Heavy Trucks on Concrete Pavement in Canada, 2004)*. In another Study conducted in 1982, by Zaniewski under Federal Highway Administration (FHWA) (*Zaniewski, 1982)* and National Research Council of Canada, Centre for Surface Transportation Technology (NRC-CSTT) *(Taylor, 2000)* it was shown that the savings in fuel is to the order of 11% to 20%. Concrete's rigid design reduces road deflection and corresponding fuel consumption.

d) **Saving on lighting**: Concrete being naturally of a lighter color, will deflect light from a vehicle or lamppost better than the darker asphalt pavement (Ref Figure 1.3). *(CAC, 2002)*

Concrete surface lighted



Bituminous surface lighted

Figure 1.3: Lighted concrete and bituminous surface (*Benefits of concrete highway, 2002)*

e) **Noise levels:** The noise levels of a concrete pavement are higher than that of flexible pavement. Studies have shown the increase in noise levels is to the order of about 2 – 4 decibels *(Nova Scotia Transportation and Public Works, 1999)*.

f) **Initial costs:** The initial cost of construction of a concrete pavement is higher than that of a flexible pavement the maintenance of a concrete pavement is lower and the design period is up to 25 – 30 years which to an extend offsets the higher initial costs (*CAC, 2002*).

g) **Suitability of weather:** Concrete pavements are better suited for high rainfall areas and colder climes where bituminous pavements tend to fail quickly. Also construction of concrete pavements can take place even during moist condition while in case of flexible pavements the weather and the surface have to be absolutely dry for pavement construction.

1.3 Scope and Organization

The Indian specification on concrete pavements are dealt with in IRC:58-2002. As per the design concrete of high strength to the order of 40 — 45MPa and a large thickness consisting of about 330 – 350mm of pavement quality concrete and 150mm of dry lean mix concrete is provided to cater to a design period of 20 – 30 years. A brief description of the rigid pavements being constructed in India under the aegis of NHAI is given in Appendix 'A'. Studying the various design criteria followed the world over an effort is made in this work to carry out a comparative analysis of the existing guidelines for design of concrete pavement and highlight the areas in which we need to strengthen our design philosophies. Analysis of stresses generated in rigid pavements has been carried out to find a more economical model.

The subsequent chapters are organized as under: Chapter 2 deals with the fundamental principles of design of concrete pavement and the various design guidelines existing in different countries. A comparative analysis is carried out based on a sample

problem and improvement suggested for Indian guidelines in Chapter 3 and structural analysis of various models of concrete slab carried out in Chapter 4, with the concluding remarks being given in Chapter 5.

Chapter 2

Fundamentals of jointed plain concrete pavement design

2.1 General

A general description of the rigid pavement along with the concept on which the pavement design is modeled is given in this chapter. A brief outline of the issues involved in design of the rigid pavements is presented. A summary of the provisions of some of the international standards related to these issues is also discussed in this chapter, though it should be borne in mind that no individual standard covered all the aspects.

2.2 Concept

A schematic representation of the jointed plain concrete pavements, and its various constituent elements such as dowel and tie bars, transverse and longitudinal joints, is shown in Figure 2.1. The pavement consists of a sub grade and sub base which builds up the bearing capacity of the soil to reasonable magnitude facilitating economical construction of the pavement. The rigid pavement consists of jointed concrete slab where the transverse joints are provided to facilitate longitudinal contraction while the longitudinal joints are provided where pavement widths are in excess of 4.5m to allow for transverse contraction and warping. The dowel bars help in the load transfer across the transverse joints while the tie bars prevent opening up of longitudinal joints due to heavy traffic, expansive sub grades etc. The concrete surface as such is smooth and hence surface texturing is carried out to enable better grip of vehicles onto the road.

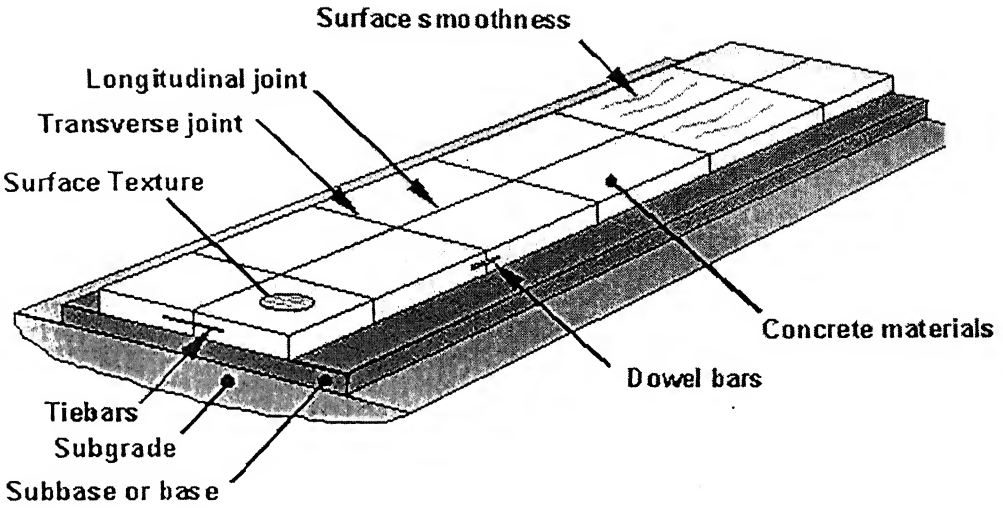


Figure 2.1: Layout of a concrete pavement *(AASHTO, 2002)*

Concrete pavement during its life period is subjected to a combination of warping, foundation restraint and traffic induced stresses which can be both compressive and flexural in nature (*David Croney and Paul Croney, 1991).* The maximum tensile stress which develops in concrete on loading primarily determines whether or not a slab cracks. In cases when the ratio of flexural stress to flexural strength is more than 0.5, it is likely to lead to progressive deterioration fatigue life of concrete and this aspect has a major impact on the thickness of the slab designed. Structurally, concrete pavement acts like a plate on an elastic foundation and as such the maximum stresses are experienced at the bottom of the slab as shown in Figure 2.2 *(Timoshenko and Woinowsky-Krieger, 1959).* When the tensile stress due to effects of warping and traffic, experienced at the bottom of the slab, exceeds the modulus of rupture of concrete at that time, cracking sets in. Increasing thickness is one way to prevent such cracking under loading. Also concrete with a higher modulus of rupture will be able to withstand the tensile stresses developed on loading of the slab. The flexural strength of concrete is related to the compressive strength by the relation *(IS 456-2000)*

$$f_{cr} = 0.7 \times \sqrt{f_{ck}} \tag{2.1}$$

where f_{cr} is the flexural strength and f_{ck} is the characteristic compressive cube strength of concrete.

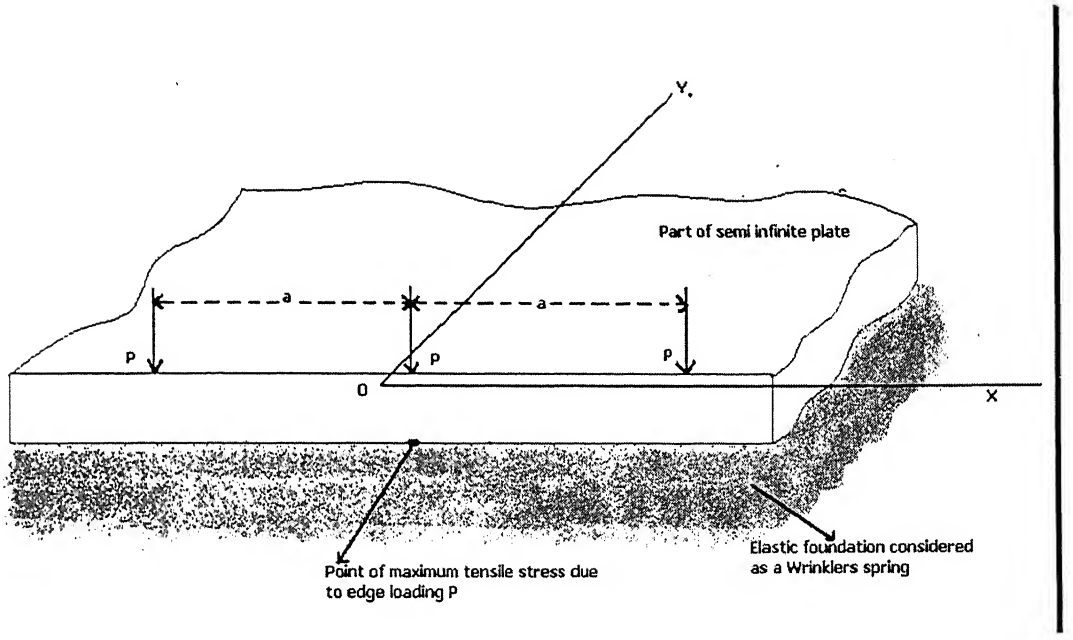


Figure 2.2: Schematic diagram of the pavement slab model

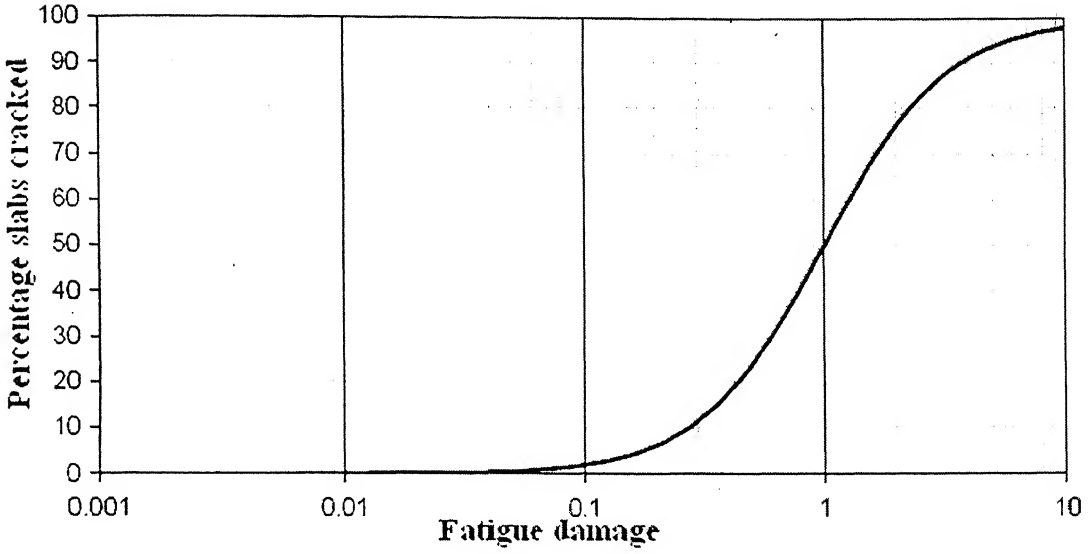
Tensile cracking can thus be resisted by both increasing the thickness of the concrete pavement and using high strength concrete which increases its modulus of rupture. Pavements experience cyclic loading and unloading over a range of axle loads during the life of the pavement which leads to quick progression of the cracks once they set in. Factors leading to concrete pavement deterioration include: heavy loads imposed by trucks, stresses induced by temperature changes, free water retained in the pavement structure and loss of sub grade support due to pumping. Cracking is directly related to the stress levels and fatigue loading experienced by the pavement. Figure 2.3 shows the relation between the fatigue damage the pavement is exposed to over the design period and the percentage of slabs cracked at the end of the design period. Thus it is seen that cracking sets in when fatigue damage exceeds 0.1 and 50% of the slabs are cracked when the fatigue damage is 1. The various types of cracking are (Ref Figure 2.4):

a) Transverse Cracking - Occurs at right angles to the centerline.

b) Longitudinal Cracking - Generally runs parallel to the centerline.

c) Corner Cracking - Intersects both longitudinal and transverse joint.

d) Intersecting Cracks (Sometimes referred to as a shattered slab) - Occurs when one or more of the different types of cracks connect or cross within a slab.



Relation between fatigue damage and slab cracking

Figure 2.3: Ratio of percentage slabs cracked to bottom up fatigue damage *(AASHTO, 2002)*

The distress types in rigid pavements are joint faulting and transverse cracking in jointed plain concrete pavements (JPCP), and punch outs in continuously reinforced concrete pavements (CRCP). In JPCP the cracks can be either be bottom up cracking or top down cracking.

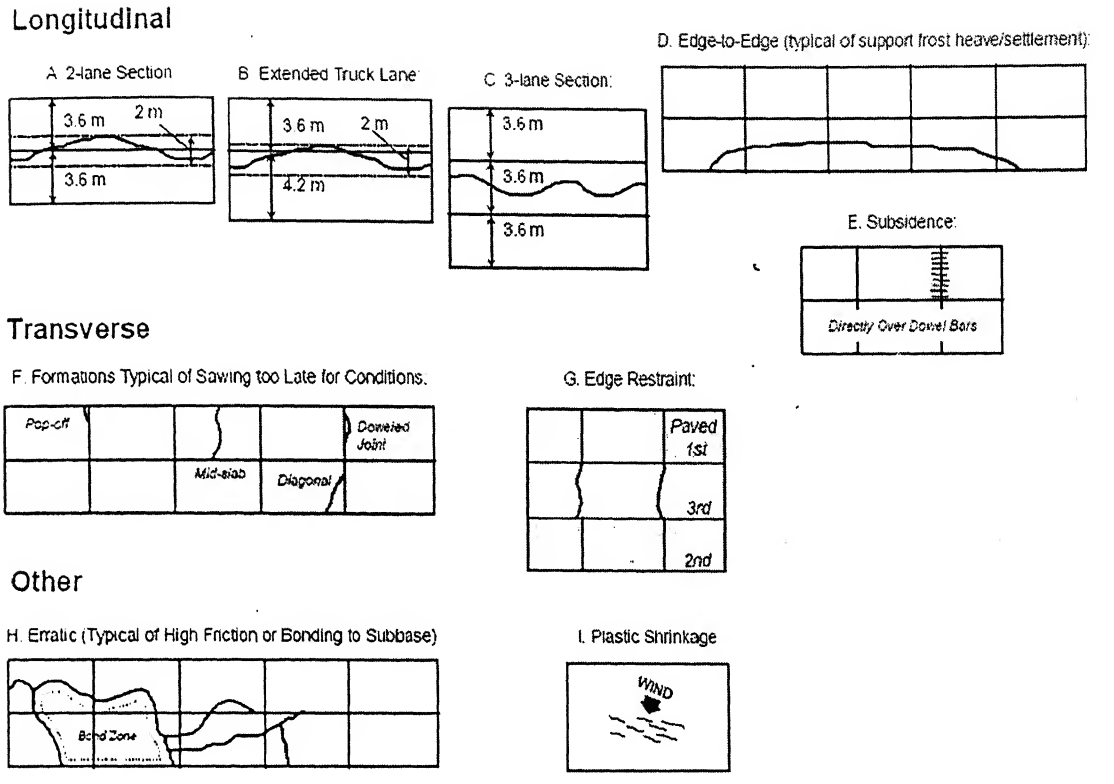
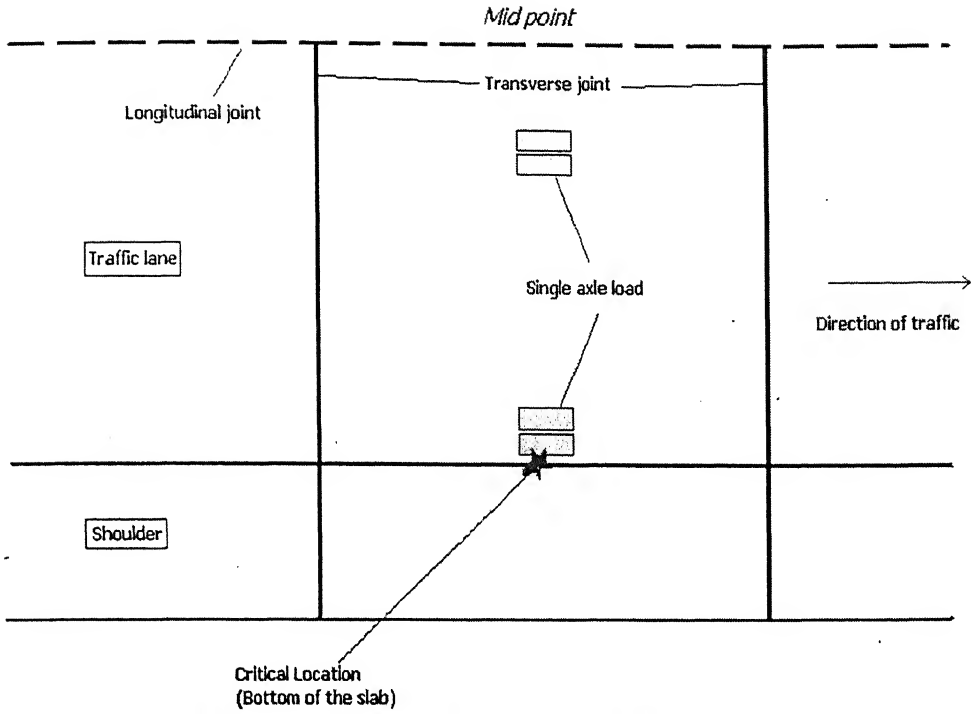


Figure 2.4: Various types of cracking in concrete pavements

2.2.1 Bottom up cracking

When the truck axles are near the longitudinal edge of the slab, midway between the transverse joints, a critical tensile bending stress occurs at the bottom of the slab which further increases when there is a high positive temperature difference through the slab. Figure 2.5 gives a schematic diagram of the loading which causes this type of cracking. Repeated loadings of heavy axles result in fatigue damage along the bottom edge resulting in micro-cracks that propagate to the slab surface and transversely across the slab *(AASHTO, 2002)*. Over time the pavement deteriorates and causes roughness leading to requirement of repairs.



Critical load and structural response location for JPCP bottom up transverse cracking

Figure 2.5: Critical loading causing bottom up cracking *(AASHTO, 2002)*

2.2.2 Top down cracking

When a truck steering axle is near the transverse joint with the drive axle about 3 to 6 meters away and still on the same slab, a high tensile stress occurs at the top of the slab between the axles, some distance from the joint which further increases when there is a negative temperature difference through the slab, a built-in negative gradient from construction, or significant drying shrinkage at the top of the slab. The sort of loading that leads to such a situation is given in Figure 2.6. Repeated loadings of heavy axles result in fatigue damage at the top of the slab, which eventually results in micro-cracks that propagate downward through the slab and transversely or diagonally across the slab which deteriorate over time leading to distress *(AASHTO, 2002)*.

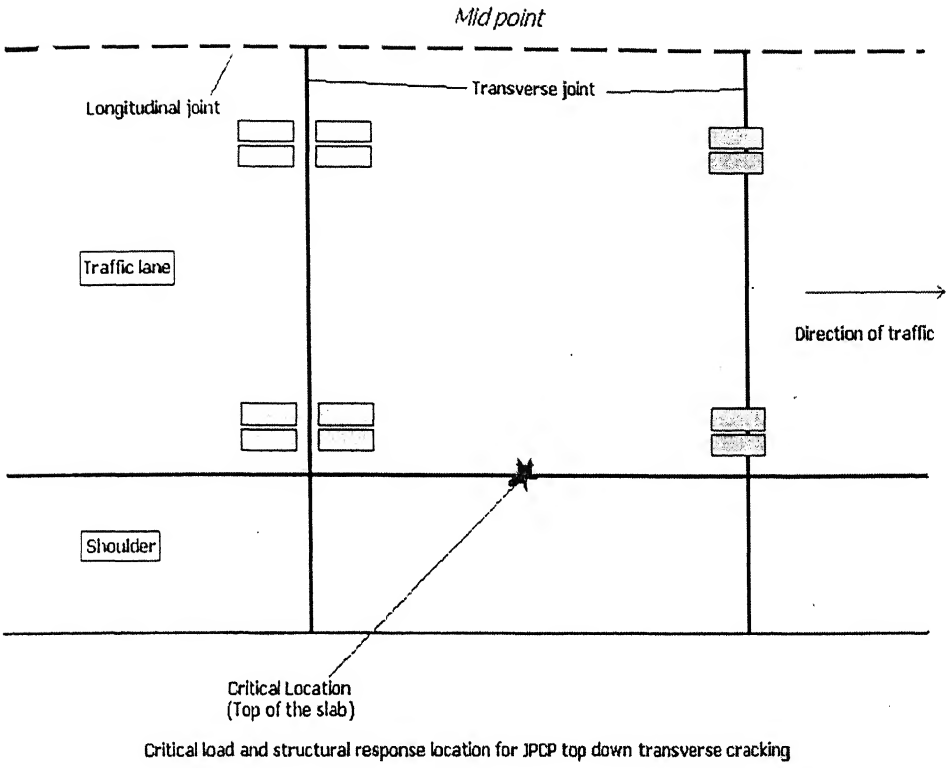


Figure 2.6: Critical loading causing top down cracking *(AASHTO, 2002)*

2.2.3 Joint faulting:

Repeated heavy axle loads crossing transverse joints create the potential for joint faulting to occur as shown in Figure 2.7. Faulting can become severe causing loss of ride quality and require early rehabilitation if any of the following conditions occurs *(AASHTO, 2002):*

a) Less than 80 percent load transfer efficiency (LTE).

b) An erodible base, sub base, shoulder, or sub grade.

c) Free moisture beneath the PCC slab.

Faulting can be controlled by providing dowels, increasing dowel size, widening slabs, providing less erodible sub base, improve large aggregate properties and providing tied shoulders.

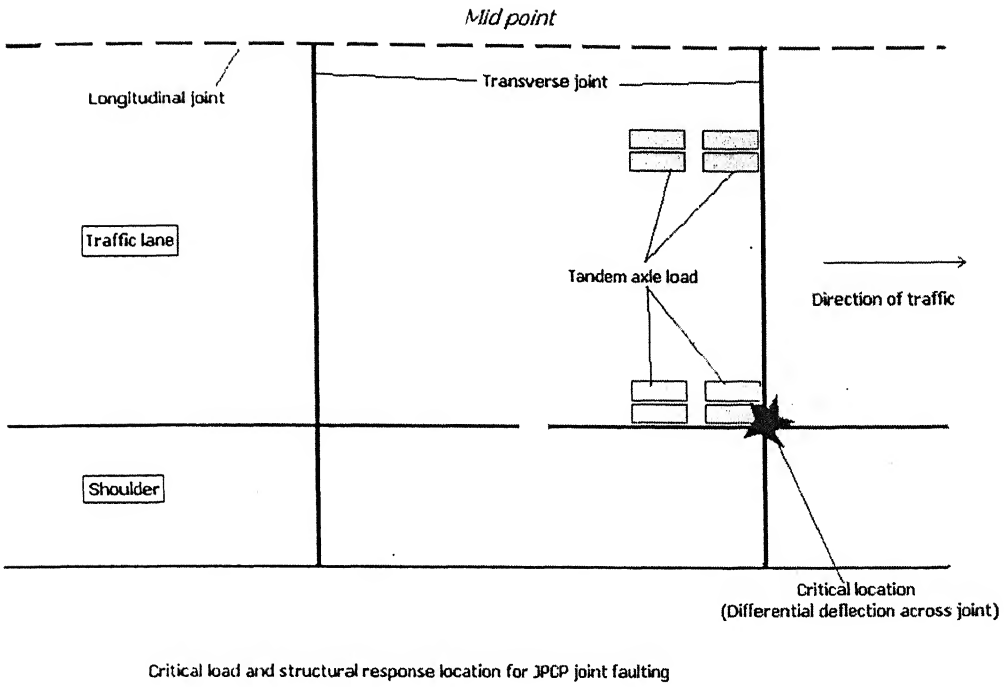


Figure 2.7: Critical loading causing joint faulting

2.3 Design factors of rigid pavement design:

Various factors are involved in the design of pavement such as climate, drainage, materials used, traffic, analysis dealing with the failure modes of the pavement. However the factors can be broadly classified into traffic loading, environment, materials and failure criteria. The failure criterion consists of fatigue failure and erosion failure. The most extensive and exhaustive research into the interplay of these factors in the performance of the rigid pavement has been carried out by AASHTO. These factors are discussed in the subsequent sections with emphasis on the methodology used in the following specifications.

a) IRC: 58-2002 (Indian Roads Congress)

b) AASHTO Guidelines (American Association of State Highway and Transport officials)

c) AUSTROADS (National Association of Road Transport and Traffic Authorities in Australia)

d) PCA 1984 (Portland Cement Association)

e) Design manual of roads and bridges (TRRL)

2.4 Design Traffic

The traffic factor consists of axle loads, number of repetitions, tyre contact area and speed of vehicle. The number of repetitions which is the cumulative traffic is worked out based on present traffic and rate of growth over the design period. In the case of Indian guidelines *(IRC 58-2002)*, Australian guidelines *(AUSTROADS, 2004)* and Portland Cement Association *(PCA 1984)* the variable axle method is used where the different axle loads are considered individually, and the 'contribution' to the cumulative damage is simply added on the basis of the number of repetitions likely for each load level. On the other hand, the American guidelines *(AASHTO, 2002)* and the British guidelines *(Design manual of Roads and Bridges, 1999)* follow the constant axle method where the different axle loads are converted to an Equivalent Standard Axle Loads (ESALs) based on the load equivalency factors. These factors are ratios developed based on experiments conducted to study the effect of damage to pavement due to passes of loads of varying magnitude and number of passes required to cause comparative damage by a standard axle load vehicle.

2.5 Fatigue failure analysis

Fatigue failure of concrete can cause both transverse cracking, which initiates midway between transverse joints on the pavement edge and longitudinal cracking which initiates in the wheel paths at transverse joints. It is seen that the critical stresses are the

flexural stresses caused by the wheel load on the pavement. The stress ratio defined as the ratio of the flexural stresses due to load and the permitted flexural stress, plays an important part in the fatigue behavior of concrete. This ratio allows the quantification of the allowable repetitions for a particular axle load. The damage ratio is found out, which is the ratio of the number of repetitions of a particular axle load during the design period of the pavement to the permitted repetition of that axle load for the stress ratio it causes. The sum of the damage ratios of the various axle loads the pavement will be subjected to in its design period gives the cumulative damage, which should be less than one or 100% depending on the mode of calculation. This cumulative damage concept is used in the fatigue analysis. A brief description on the analysis by various guidelines is given here.

2.5.1 Guidelines for Design of Plain Jointed Rigid Pavements *(IRC: 58-2002)*

It has been mentioned above that IRC guidelines consider the axle loads separately, (and not as an equivalent axle load). Now, based on a study on lateral placement of commercial vehicles, they provide for 25% of the cumulative traffic over the design period to be considered for design *(Reddy and Pandey, 1992)*.The rigid pavement is analyzed for maximum likely flexural stress level the pavement will be subjected to due to vehicle loads, warping and corner stresses. The maximum load stresses are worked out from the Westergaard's equation modified by Teller and Sutherland.

$$\sigma = 0.529\frac{P}{h^2}(1+0.54\mu)\left[4\log_{10}(l/b)+\log_{10}(b)-0.4048\right] \qquad (2.2)$$

where,

σ load stress in the edge region

P design wheel load

h pavement thickness in cm

μ Poisson's ratio for concrete

E modulus of elasticity of concrete, kg/cm^2

k Modulus of sub grade reaction, kg/cm^3

b radius of equivalent distribution of pressure

a radius of load contact area assumed to be circular.

2.5.2 Guide for Design of Pavement Structures, 1993 (AASHTO) (*AASHTO 1986, 1993)*

The analysis is based on empirical equations developed from AASHTO road test with modifications based on theory and experience. The design equation includes the effect of the various parameters discussed above and is given by:

$$\log W_{18} = Z_R S_0 + 7.35\log(D+1) - 0.06 + \frac{\log\left[\Delta PSI/(4.5-1.5)\right]}{1+1.624\times10^7/(D+1)^{8.46}} + (4.22-0.32p_t)\log\left\{\frac{s_c C_d\left(D^{0.75}-1.132\right)}{215.63J\left[D^{0.75}-18.42\left(E_c/k\right)^{0.25}\right]}\right\} \quad (2.3)$$

where,

W_{18} load in million ESAL's

$Z_R S_o$ Reliability term where Z_R is the standard normal deviate and S_o is the overall standard deviation

D slab thickness in inches

PSI design serviceability loss

S_c Mean concrete modulus of rupture

C_d Drainage coefficient

J Load transfer coefficient

k Effective modulus of sub grade reaction

E_c Concrete elastic modulus

2.5.3 Guide for Pavement Design (AUSTROADS 2004) *(AUSTROADS, 2004)*

This is based on the likely repetitions of different axle loads during the design period of the pavement. The allowable load repetitions are found out based on the Miner's hypothesis when the stress ratio is more than 0.45. The equivalent stress is found using Equation 2.4 below,

$$S_e = a + b/T + c \times \ln(E_f) + d/T^2 + e\left[\ln(E_f)\right]^2 + f \times \ln(E_f)/T + g/T^3 + h\left[\ln(E_f)\right]^3 + i\left[\ln(E_f)\right]^2/T + j \times \ln(E_f)/T^2 \quad (2.4)$$

where,

S_e is the equivalent stress

a to j are the coefficient tabulated in the guideline

T base thickness in mm

E_f is the effective sub grade strength

2.5.4 Portland Cement Association 1984 *(PCA 1984)*

The analysis is based on the edge stress mid way between the transverse joints with the most critical loading position as shown in Figure 2.8. An average modulus of sub grade reaction over the design period is used and the edge stress is worked based on empirical equation derived from experimental data. The stress ratio is found and the permissible number of repetition worked out. Finally, the fatigue damage due to individual axle loads is summed up, and it should be ensured that it is less than unity.

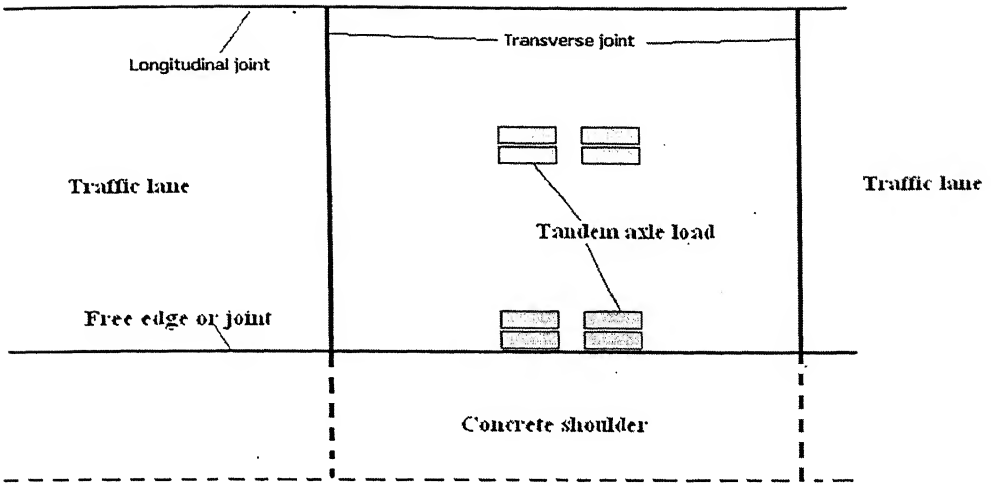


Figure 2.8: Critical loading position for fatigue analysis *(PCA, 1984)*

2.5.5 Design manual for Roads and Bridges, Volume 7 *(TRRL Report 1132)*

This design is based on fatigue analysis. The equations are based on empirical equations developed from performance of full scale experimental roads. The design is standardized for a design period of 40 years and graphically represented. The empirical relation fort he thickness of the pavement is given by the formulae

$$Ln(L) = 5.094Ln(H) + 3.466Ln(S) + 0.4836Ln(M) + 0.08718Ln(F) - 40.78 \quad (2.5)$$

Where

Ln is the natural logarithm

L is the cumulative traffic in million standard axles (msa)

H is the slab thickness in mm

S is the 28 day mean compressive strength in MPa

M is the equivalent foundation modulus in MPa

F is the percentage failed bays

2.6 Erosion failure analysis

This analysis gives the distress due to pumping, erosion of material beneath the slab and due to faulting. Erosion failure is largely caused by the action of tandem axle loads, and quantitative understanding is derived from empirical data collected from existing roads. The factors causing the erosion include presence of water, rate at which water is ejected from the sub base, the erodibility of the sub base, magnitude and number of repeated loads and the amount of deflection *(Huang, 1993)*. The AUSTROADS and PCA cater for erosion distress. The AASHTO indirectly co opts this in the empirical relations developed, in the form of loss of support of the sub base the values of which vary from 0 to 3.0 based on the material used in the sub grade. Given the small component of tandem and multi axle vehicles in India, IRC does not provide for design for this form of distress. It is however, recommended that tied shoulders are provided to mitigate the effects of erosion.

2.6.1 Guide for Pavement Design (AUSTROADS 2004) (*AUSTROADS, 2004)*

The AUSTROADS have developed an empirical formula to cater for erosion distress. The equation gives the allowable load repetition for various axle loads and the cumulative total as for fatigue should not exceed 100%.

$$\log\left(F_2 N_e\right) = 14.52 - 6.77\left[\max\left(0, \left(\frac{PL_{SF}}{4.45F_4}\right)^2 \times \frac{10^{F_3}}{41.35} - 9.0\right)\right]^{0.103} \qquad (2.5)$$

where,

F_2 adjustment for slab edge effects which is

0.06 for pavement without concrete shoulders and

0.94 for pavement with concrete shoulder

N_e Allowable load repetitions

P axle load group (KN)

L_{SF} load safety factor

F_3 erosion factor

F_4 load adjustment for erosion due to axle group

2.6.2 Portland cement Association 1984 *(PCA 1984)*

The pavement distress due to erosion is worked out in PCA 1984 on the basis of empirical equation developed from AASHTO road tests. The allowable load repetitions are found for different axle loads and their cumulative damage worked out. For a particular design thickness this should be less than 100%. The allowable load repetitions are computed based on equation 2.6.

$$\log N = 14.524 - 6.777\left(C_1 P - 9.0\right)^{0.103} \tag{2.6}$$

where,

N allowable number of load repetitions

C_1 adjustment factor with value of 1 for untreated sub bases and 0.9 for stabilized sub bases

P $= 100\sum_{i-1}^{m}\left[\frac{C_2 n_i}{N_i}\right]$ gives the cumulative erosion damage (2.7)

Where,

C_2 factor, 0.06 for no shoulder and 0.94 for tied concrete shoulder

N_i allowable number of repetitions for axle load i

n_i predicted number of repetitions for axle load

With a concrete shoulder the corner deflection is not significantly affected by truckload placement and as such a large C_2 value can be used. The percentage erosion damage, P found should be less than 100%.

2.7 Environmental effects

Environmental effects include the effects of temperature and precipitation. Temperature effects can be direct, caused by stresses induced due to variation of temperature or indirect, caused due to the freezing and thawing of the water present in the sub grade thus leading to a change in the modulus of the sub grade. The environmental effects can be dealt with under the following sub heads.

2.7.1 Swelling of soil: Precipitation leads to percolation of the surface water into the sub grade. The degree depends on the drainage conditions. This can cause swelling in expansive soils which leads to loss in riding quality and serviceability of the pavement. AASHTO design caters for these effects on the basis of swell rate constant, potential vertical rise and swell probability, and the net loss in serviceability due to environmental factors is deducted from the design serviceability.

2.7.2 Frost heave: The direct effect of temperature is the curling of slab. However in cold climate the resilient modulus of the unstabilized material is affected due to the freeze thaw cycle. This generally occurs in frost susceptible soils where water is present and the temperatures fall below freezing. In such conditions the water freezes into ice lens causing volume changes, and leading to permanent distortions on the pavement surfaces *(AASHTO, 2002)*. Frost heave leads to differential heaving, surface roughness and cracking. AASHTO caters for this effect by working out frost heave on the basis of frost heave rate, maximum potential serviceability loss and probability. Precipitation also

causes an increase in moisture content which is related to drainage, as discussed in the following paragraphs.

2.7.3 Drainage: Drainage as a factor in design is catered for empirically in AASHTO guidelines based on the observation of AASHTO road tests. Different types of entry of moisture into the pavement are shown in Figure 2.9. Inadequate drainage leads to high exposure of moisture causing problems such as *(AASHTO, 2002)*:

a) Softening of pavement layers and sub grade when they are saturated and remain so for a length of time.

b) Quality of material degrades on interaction with moisture

c) Saturation with moisture also leads to loss of bond between the pavement layers.

It should be noted that Indian guidelines provide for a drainage layer to enable quick disposal of water though the same is not directly considered in the design process

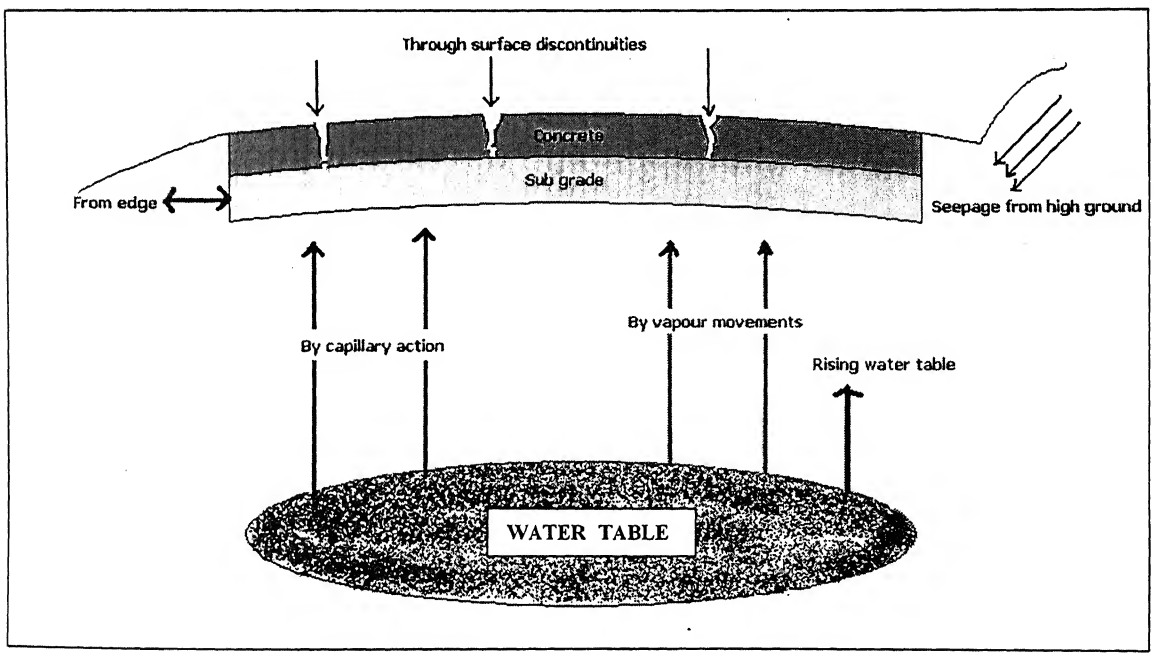


Figure 2.9: Schematic diagram showing the ingress of moisture into the pavement *(AASHTO, 2002)*

2.8 Reliability and serviceability

Reliability is a means of incorporating some degree of certainty in the design process to ensure that various design alternatives will last the analysis period. It takes into account the chance variation in traffic prediction and performance prediction. Thus there is a level of assurance which is worked into the design. When volume of traffic, difficulty of diverting traffic and public perception of availability increases, the risk of non performance should be minimized, this is accomplished by the introduction of reliability factor. Depending on reliability the safety factors are quantified using empirical relations. This factor is incorporated in AASHTO and AUSTROADS guidelines. A sample of reliability factors used in AASHTO guidelines are given in Table 2.1.

Table 2.1: Reliability factor *(AASHTO 1986, 1993)*

Functional classification	Recommended level of reliability	
	Urban	Rural
Interstate and other freeways	85 – 99.9	80 – 99.9
Principal Arterials	80 – 99	75 -- 95
Collectors	80 – 95	75 – 95
Local	50 – 80	50 – 80

The serviceability of a pavement is defined as the ability to serve the type of traffic which uses the facility. This index is used by the AASHTO guideline which is based on a serviceability-performance concept. AASHTO present serviceability index varies from 0 (impossible road) to 5 (perfect road). Selection of the lowest or terminal serviceability index is based on the lowest index that will be tolerated before rehabilitation, resurfacing or reconstruction becomes necessary. Studies are conducted on the user satisfaction levels on the nature of the road and then the index is related empirically as given in Table 2.2.

Table 2.2: Design Serviceability factor *(AASHTO 1986, 1993)*

Terminal serviceability level	Percent of people stating unacceptable
3.0	12
2.5	55
2.0	85

2.9 Summary

A brief description of the effect of various factors in the design of rigid pavement and how the various guidelines have incorporated these factors in their design have been discussed in this chapter. A comparison of the various guidelines viz the Indian guideline along with a model design exercise to quantify the variation in the different design concepts is carried out in the next chapter.

Chapter 3

Comparing different guidelines

3.1 General

The previous chapter provided details of some of the important issues relating to the design of rigid pavements. It should be noted that not all aspects are addressed in any specification. An effort has been made in this chapter to provide a comparative analysis of different specifications and also to carry out a model analysis for a hypothetical pavement using realistic data following the procedures suggested by the different guidelines. The exercise has been carried out to identify the areas where the Indian specifications need to be strengthened. Some suggestions for such remedial measures are also made.

3.2 Summary of available guidelines

Though several countries and professional organizations have published specifications relating to rigid pavements, the following have been used in this thesis for study and reference, considering their availability and extent of use.

a) IRC: 58-2002 (Indian Roads Congress)

b) AASHTO Guidelines (American Association of State Highway and Transport officials)

c) AUSTROADS (National Association of Road Transport and Traffic Authorities in Australia)

d) PCA 1984 (Portland Cement Association)

e) Design manual of roads and bridges (TRRL)

The following paragraphs give a comprehensive summary of the methodology adopted in the above specifications.

3.2.1 AASHTO guidelines

The AASHTO guidelines comprising of Volume I which was published in 1986 and Volume II, published in 1986 and revised in 1991 is used in the study here. A new version of the guidelines was prepared in 2002, and is presently in the process of being finalized. The AASHTO guideline is based on a mechanistic empirical model. The approach is based on the application of structural models to calculate pavement response and using this response to predict distress models based on laboratory tests and field performance data. It is based on the fatigue cracking model where the stresses due to wheel placement and curling are considered. The cumulative design traffic is converted into ESALs using traffic equivalency factors arrived at based on AASHTO road tests. Extensive tests were carried out to predict distress due to varying drainage, load transfer between joints, loss of support due to erosion of sub base, and other factors such as soil swelling, frost heave, etc. Reliability factors are introduced based on the standard deviation in the values of the test results and the levels of user satisfaction.

3.2.2 PCA 1984

The PCA 1984 carries out both fatigue and erosion analysis. It uses variable axle loads to arrive at the critical stresses for further analysis. Load transfer in the form of dowels or aggregate interlock and provision of concrete shoulder is taken into consideration in the design. Load transfer had an effect on the erosion distress of the pavement as it affects the deflection of the pavement while the provision of shoulder affects both the fatigue behavior and the erosion behavior of the pavement.

3.2.3 AUSTROADS guidelines

The AUSTROADS guideline is based on PCA 1984 method with suitable modifications to suit the Australian conditions. It is based on the variable axle method where the actual axle loads are used to predict the stresses, which are then used to carry out the fatigue analysis. The design incorporates erosion analysis and also takes into consideration the provision of load transfer devices and concrete shoulders. The overall safety factor is introduced in the form of project safety factor, and axle loads are multiplied by this factor to work out the design traffic.

3 2.4 Design manual for roads and bridges (TRRL)

The British guidelines given in the design manual for roads and bridges is based on the fatigue failure of the pavement. Standardized design curves are provided which are based on statistical evaluation of empirical data from 29 unreinforced and 2 reinforced concrete highway pavements which was collected in the mid 1980's. The daily traffic is converted into cumulative ESALs for a design period of 40 years. Provision of concrete shoulder is considered in the design.

3.2.5 IRC: 58-2002

The Indian guideline is based on a mechanistic approach. The critical part of the design is the fatigue failure analysis. A check is carried out for the stresses developed due to temperature and corner loading. As the number of tandem and multi axle vehicles which cause erosion distress of the pavement, is quite small, the document does not address erosion analysis. Design is based on the variable axle method where the flexural stresses of each axle load is found to work out the fatigue damage due to the cumulative

number of repetition of that load. The guideline provides for the provision of a drainage layer for quick disposal of water likely to enter the sub grade and recommends the provision of tied concrete shoulders. However the impact of providing or not providing these on the design is not considered. The strength of the sub grade is considered in the terms of soil modulus which is found from the plate test or converted from the CBR test results. The effect of environment is considered in as much as the minimum soil modulus got is taken for design and this is during or immediately after the monsoons when the worst moisture conditions are experienced. A safety factor based on the truck volumes the road is likely to experience is introduced as in the case of PCA and AUSTROADS. This is basically to cater for unpredicted heavy truck loads. Based on a study on lateral placements of vehicular traffic in Indian Highways where it was seen that only 25% of the traffic travel on the outer edge of the lane generating critical stress and thus only 25% of the design traffic is taken in the analysis of the slab.

3.3 Comparison of guidelines

A comparison of the various guidelines has been given in a tabulated form in Table 3.1. It can be seen that no single guideline addresses all the issues as mentioned earlier also. A comparison of the salient feature of the various guidelines shows that in general, the basis of all the guidelines is the fatigue analysis to check whether the concrete thickness provided is capable of sustaining the load repetitions likely to occur over the design period. The magnitude of load and the repetitions is found out from the design traffic. The details of the methodology adopted by various guidelines for working out the cumulative traffic during the design period are however, different. For example, the AASHTO and TRRL guidelines convert the different axle loads into an equivalent axle load, whereas the others consider individual axle loads separately. In the former case,

experimental data collected for converting various axle loads to an ESAL are based on various factors such as the concrete properties, the quality of workmanship, the soil type and compaction achieved, the type of load applied, etc,. A proper understanding of the interdependence of these factors and their effect on the performance of the pavement is required for extrapolating these results to arrive at a range of equivalency factors for various scenarios. The AASHTO guideline is the only one based on the serviceability design concept where the pavement distress is quantified into acceptability norms. Here the opinion of road users and the effect of environmental factors such as soil swelling, frost heave are taken into account to formulate a scale which is related to the other design aspects of the pavement. All the specifications carry out the fatigue analysis to arrive at the damage likely to be caused by repeated loading. However the erosion analysis is carried out by only the Australian and PCA specifications. The AASHTO specification considers the affect of loss of support in the design.

Table 3.1: Comparison of various guidelines

Guideline	Reliability	Serviceability	Design Traffic	Frost heave	Drainage	Fatigue Analysis	Erosion Analysis
Guidelines for Design of Plain Jointed Rigid Pavements (IRC: 58-2002)	Not designed for	Not designed for	Cumulative traffic over design period considered (only 25% taken)	Not designed for	Not designed though drainage layer recommended	Based on Miner's Hypothesis. Critical stress found on the basis of Westergaard's equation modified by Teller and Sutherland	Since caused largely by tandem and multi axle vehicles not designed for due to their small numbers on Indian highways
Guide for Design of Pavement Structures, 1993 (AASHTO)	Considered. Varies from 0 to 5	Considered depending on the importance of the road	Traffic under various axle loads converted to an equivalent standard axle load.	Considered.	Depends on time to drain water and percentage of time exposed to saturation moisture levels	Based on empirical equations developed from AASHTO road test with modifications based on theory and experience	Partially considered in that loss of support taken into consideration for design.
Guide for Pavement Design (AUSTROADS 2004)	Considered	Not considered	Cumulative traffic under various axle loads worked out	Not considered	Not considered	Critical stresses found and fatigue damage worked out based on Miner's hypothesis	Designed to take into account the sub grade/sub base erosion arising from repeated deflection at the planned cracks
Design manual for Roads and Bridges, Volume 7 (TRRL Report 1132)	Not designed for	Not designed for	Traffic under various axle loads converted to an equivalent standard axle load	Not considered	Not considered	Based on empirical equations developed from performance of full scale experimental roads	Not considered
Thickness Design of concrete pavements, PCA 1984	Not designed for	Not designed for	Cumulative traffic under various axle loads worked out	Not considered	Not considered	Critical stresses found and fatigue damage worked out based on Miner's hypothesis	Considers the erosion of foundation due to corner deflection developed empirically from AASHTO road test

3.4 Illustrative example

3.4.1 General

As a means of quantifying the designs output of various guidelines and thus compare to arrive at a better understanding of the positive and negative aspects of the provisions of the Indian guidelines a model analysis was carried out based on a constant set of input parameters. The requirements of various guidelines are tailor made to suit the conditions present in the parent country. However in case of any lack of data or discrepancy necessary data available from the guidelines and suitable assumptions have been made and same mentioned in the calculations provided here. A representative example has been solved using some of the different design guidelines, including IRC 58:2002.

3.4.2 Input data

Though the procedure to be followed for arriving at the thickness of the concrete pavement using different specification has been briefly explained above, a sample problem using an identical basic data set has been worked out, and the results discussed in this chapter. A comparative analysis is carried out based on a common set of input variables as given here. Where there is a variation, effort is made to complement the Indian scenario to the extent possible. It should be pointed out that the data given here is for a cement concrete two-lane two-way highway in Karnataka, and has been used for an example in IRC 58-2002. The following are the salient features of the data used as input.

a) Modulus of sub grade:

i.	Modulus of sub grade soil reaction	8.00 kg/cm^3
ii.	Thickness of DLC	150mm

b) Properties of cement concrete

i. Design flexural strength of concrete (28 days) — 45 kg/cm^2

ii. Modulus of elasticity of concrete — 3.0×10^5 kg/cm^2

iii. Poisson's ratio — 0.15

iv. Coefficient of thermal expansion — 10×10^{-6} / °C

c) Traffic factors

i. Number of commercial vehicles (both directions) — 3000

ii. Traffic growth rate per annum — 7.5 per cent

iii. Tyre pressure — 8.0 kg/cm^2

d) Load safety factor — 1.2

e) No: of lanes — 2

f) Design period — 20 years

g) The axle load distribution is given in Table 3.2.

Table 3.2: Assumed axle load distribution

Single Axle Loads		Tandem Axle Loads	
Axle load class	**Per cent**	**Axle load class**	**Per cent**
19-21	0.6	34-38	0.3
17-19	1.5	30-34	0.3
15-17	4.8	26-30	0.6
13-15	10.8	22-26	1.8
11-13	22.0	18-22	1.5
9-11	23.3	14-18	0.5
<9	30.0	<14	2.0
Total	93.0	Total	7.0

In addition to the above, values for the following were required for determining the thickness of the pavement as per other guidelines. The values used were chosen in a manner that it reflected the Indian conditions. This has been done to ensure that the

thickness is arrived at for a set of conditions that are as similar as possible, to enable equitable comparisons.

a) Load Transfer coefficient: 2.5 from a range of 2.5 – 3.1. A higher value may be taken when dowels and shoulders are not provided.

b) Drainage coefficient: 1.1 from the range of 1.0 – 1.3, given that IRC specifications advocate the provision of a drainage layer and the exposure to saturation level moisture in Karnataka is not likely to be more than 25% of the time, and hence 1.1 has been used.

c) Design serviceability factor: 2.0. This value is the difference between an initial serviceability index of 4.5 and a terminal serviceability index of 2.5 (which can vary). The latter is 2.5 for major highways and 2.0 for highways of lesser traffic volumes though the former is kept at 4.5[11, 12].

d) Reliability factor: 95% from a range of 85% – 99.9%. The value may be decreased for roads of lesser importance. It may be noted that both AASHTO and AUSTROADS use the 95% level in the case of highways.

e) Equivalent foundation modulus. For the built up sub grade modulus of 8kg/cm^3 the likely combination of underlying layers is from the table given in IRC a sub grade of 2% CBR and a sub base course of DLC 10cm thick A value of 270 MPa corresponding to this combination is taken (TRRL)[18].

Similarly, the traffic was converted to ESALs as per the AASHTO/TRRL requirement. Also since the Indian specifications recommend the provision of concrete shoulder, the corresponding values were taken for AASHTO, AUSTROADS and TRRL analysis. The following paragraphs outline the methodology used by the different specifications in arriving at the design thickness of pavement.

3.4.3 Discussion of results

Calculations for the required pavement thickness were carried out using the above input data in accordance with the various guidelines such as IRC: 58-2002, AASHTO, AUSTROADS and British guidelines. The details of the calculations are given in Appendices 'B', 'C', 'D' and 'E' respectively. The results are discussed in this section.

An analysis of the results show that the analyses by the Indian design specifications result in thickness which are 10-14% higher than the other guidelines. Table 3.3 summarizes the thickness of rigid pavements as determined using the different specifications. The design thickness of 330mm for the Indian codes is the highest, even though the design traffic load is the least. Reducing a centimeter of thickness in a 7m wide road, could result in a saving of Rs 2.4 lakhs per kilometer of pavement, and several thousands of kilometers of pavement are in various stages of planning, it is very important that a closer look at the design parameters and specifications is taken at the earliest.

Table 3.3: Pavement thickness obtained from case study

Design Guidelines	Pavement Thickness (mm)	Cumulative Traffic	Design Traffic
IRC 58:2002	330	47 million axles	11.69 million axles (1)
Design of Pavement Structures, 1993 (AASHTO)	300	252.63 ESALs	126.32 ESALs (2)
Guide for Pavement Design (AUSTROADS 2004)	290	41 million axles	9.32 million axles (3)
Design manual for Roads and Bridges, Volume 7 (TRRL Report 1132)	300	200 ESALs	150 ESALs (4)

Notes:

1. The IRC, AASHTO and the AUSTROADS have a design period of 20 years while the design period in TRRL is 40 years.

2. For Indian specifications, the design traffic is worked out on the basis of the cumulative axle load over the design period of which 25% is taken as the design traffic on the basis of a study on the lateral placement of vehicles moving on a highway (1). Thus the design traffic is taken as 11.69 million axles, which is 25% of 47 million axles.
3. AASHTO specifications convert the various axle loads into ESALs (2). The design traffic is half of the cumulative traffic as the daily traffic is taken in both directions.
4. In AUSTROADS specification the cumulative traffic accounts for a single direction traffic and the design traffic on which the likely repetitions of axle loads is found out is based on the proportion of individual axle load groups. This figure comes to 9.242 million axles (3).
5. For the TTRL the design traffic is made available from ready made design charts and as such gives the final figures after taking into account all the factors to be considered as per that specifications.

It is seen that in Indian specifications though the analysis is based on the same principles as the other design guidelines the results arrived are more conservative. Studies have to be carried out to better understand the inter relation of various parameters affecting the performance of the pavement, thus enabling economical and relevant design. Some of the areas which require special focus have been discussed briefly in the following section.

3.5 Strengthening IRC 58: 58-2002

The following are some of the areas where indigenous data needs to be collected or generated, either through appropriate research in laboratories or collection from the field. In case there is a need to instrument the pavement, which may also be possible at this stage as most of the concrete pavements are being built.

a) **Erosion distress:** This occurs due to deflection of the pavement on loading by tandem and multi axle loads which leads to pumping and erosion of the sub base. The effect of loss of support or erosion analysis is not catered in the Indian design as the percent of traffic consisting tandem and multi axle load is small. How ever the point to be noted is that the pavements are designed for a period of 20-30 years during which the mix of traffic will change vastly.

b) **Reliability factor:** A study is to be carried out to better relate the safety factor to the nature and importance of the road as well as the attitude and psychology of road users.

c) **Provision of concrete shoulders:** Provision of shoulders lead to the pavement experiencing less stresses as also preventing distress caused by loss of support due to erosion of sub base. The Indian design recommends the provision of tied concrete shoulders but does not cater for the reduction in thickness this will accrue due to the pavement experiencing reduced stresses. Studies need to be carried out to incorporate this aspect in the design.

d) **Drainage:** The presence of moisture affects the modulus of the sub grade. Water leads to pumping and degradation of paving materials and provision of drainage layer prevents the ingress of moisture into the pavement system which will have an effect on the pavement design. Indian specification calls for the

provisioning of drainage layer depending on the high flood level, though the effect of provisioning or non-provisioning of this is not considered in the design.

e) **Environmental effect:** The American, Australian and the British guidelines analysis are based on experimental analysis used to calibrate the distress of pavement structure modeled on mechanistic analysis. This helps in better understanding the inter relation between various factors such as moisture, temperature, soil swelling, frost heave, etc which affect the performance of pavement and their variation with the change in terrain and climate across the country. These should be incorporated to enable analysis and design of pavement suiting the local conditions and requirements.

f) **Analysis:** Research should be carried out to find out the effect of incorporating the shoulder as an integral part of the pavement slab as well as having variable thickness of slab as the present analysis caters for same thickness arrived at by analysis for critical loading at the edges. The slab hardly experiences these stresses for the same loading at the interiors.

3.6 Summary

Almost all the above areas to be considered other than the stress analysis of pavement structure require formulation of empirical relation by evaluation of data collected from experimental studies. As such study of the same is beyond the scope of this thesis. It is known that the critical stress experienced by the slab reduces as the load moves away from the edge of the slab towards the interior. This fact has been made use of in carrying out the stress analysis of slabs with varying thickness. The next chapter deals with the analysis of a pavement structure model where the thickness is varied with distance from the edge of the pavement and another model where the concrete shoulder

forms an integral part of the pavement structure. A sample model with the input data of an ongoing project in NHAI was used to carry out stress analysis based on the above models and the reduction in cost achieved by the adoption of the above models was worked out.

Chapter 4

Stresses in jointed plain concrete pavements

4.1 General

A basic representation of the concrete pavements has already been discussed in Chapter 2 (e.g. Figure 2.1). It should be noted that concrete pavements are often cast using a continuous paver, and then 'individual' slabs of about 3.5m X 4.5m are created by sawing longitudinal and transverse slits before the concrete sets. These cuts are made just after the initial curing of the concrete to basically induce cracks along the cut. They serve an important purpose in localizing cracking and minimizing the shrinkage and contraction cracks. Thus, structurally the analysis can be carried out considering slabs measuring 3.5m x 4.5m resting on elastic foundation and subjected to axle loads located at different locations. A schematic representation of jointed plain concrete pavement is shown in Figure 4.1 for one side of a four lane highway. It comprises of two lanes and a shoulder with the longitudinal and transverse joints as shown.

In this study, stresses at different points in the slab were computed using Structural Analysis Program (SAP2000), Advanced version 9.0.3 and varying the magnitude and location of the load, etc. It was found that the critical stresses experienced by the slab reduce steeply as the load moves to the interior and that considerable economy could be achieved by innovative designs in the slab, and also a more realistic modeling of the pavement structure. Also slabs modeled with shoulder as an integral part of the main

slab experience lower critical stresses as the lane edge loading are actually interior loading on the slab as the shoulder is integral to the slab.

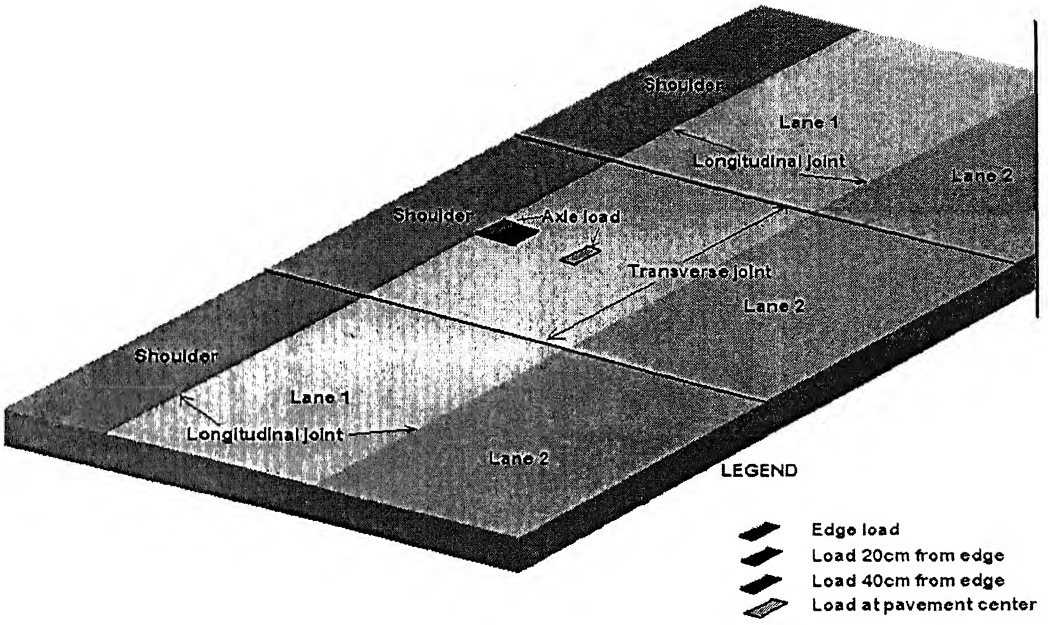


Figure 4.1 Schematic representation of the slab showing the load application

4.2 Structural model for pavement slab

For the purpose of analysis, the slab was taken to be made of plain concrete resting on elastic foundation and modeled using FEM as shown schematically in Figure 4.2. These assumptions are in accordance with the Westergaard's hypothesis that the concrete pavement slab rests on a sub grade/sub base which is considered to be an elastic foundation modeled as Wrinkler's springs. It should be noted that the stiffness of these springs should be taken in a manner that the stiffness of the sub-grade and sub-base including the dry lean concrete, is appropriately accounted for. Thus in the analysis the effective modulus of the strengthened sub base is taken.

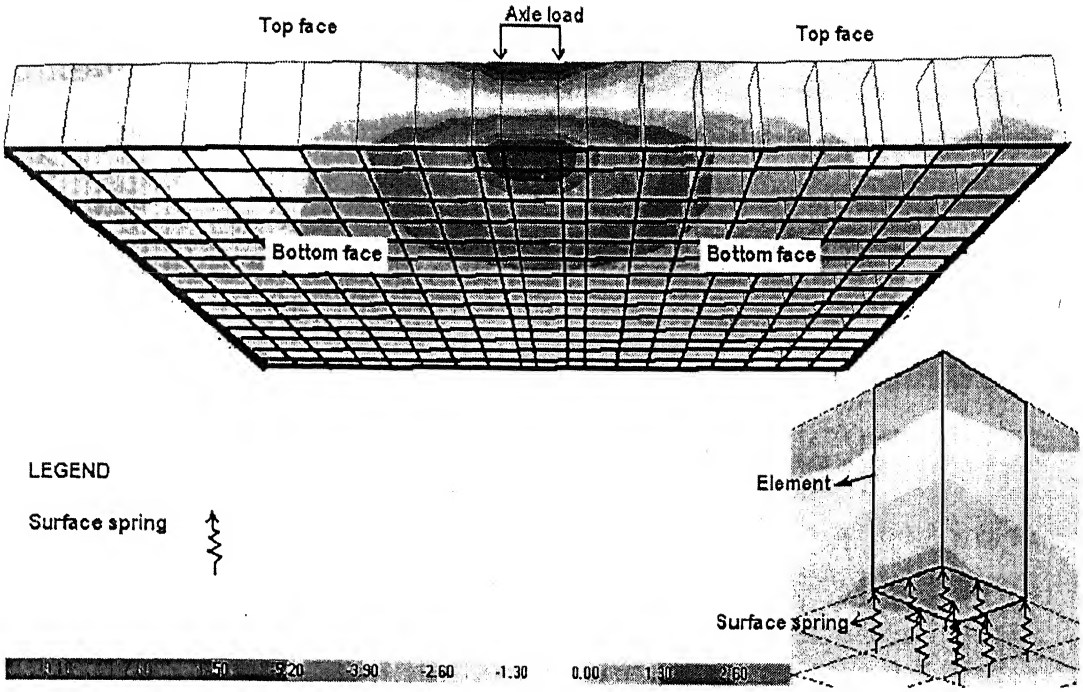


Figure 4.2: Schematic representation of a slab and an element showing the different faces

As discussed earlier also, the role of tie bar is to prevent widening of joints due to heavy traffic or expansive sub grades and that of the dowel bars is to enable load transfer. The dowels permit the lateral movement of the slab allowing for contraction and expansion while the tie bars help in keeping the slabs in position. As far as modeling is concerned, the longitudinal edges are modeled as free edges as the tie bars are provided only to prevent the longitudinal joints from widening. The dowel bars provided at the transverse edges help in load transfer and permit the movement due to contraction and expansion of slab. Thus transverse edge is modeled to permit only the longitudinal movement of the slab. The positioning of the dowel bars and tie bars are shown in Figure 4.3.

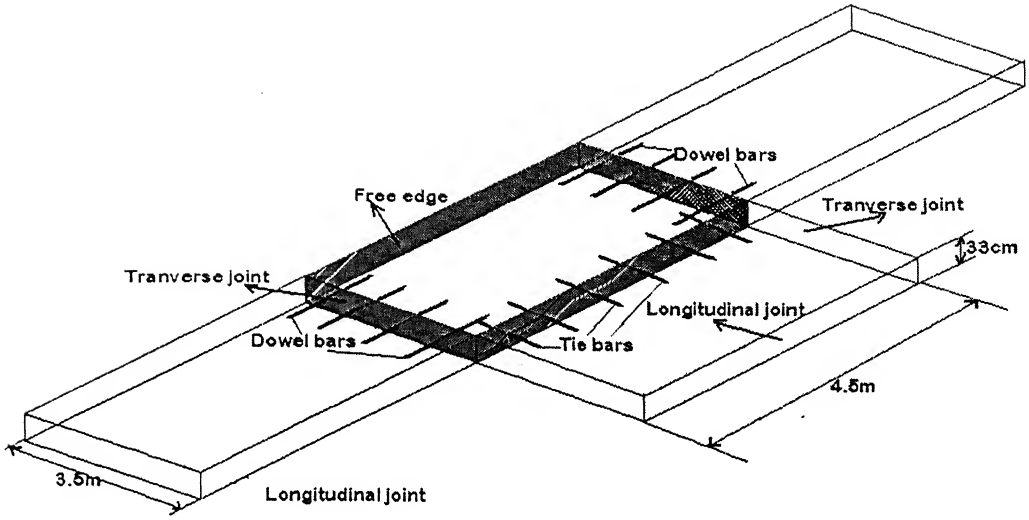


Figure 4.3: Schematic representation of the pavement showing the dowel and tie bars

In this study, the slab was modeled using eight node elements measuring 20cm×25cm×33cm. Initially various element sizes were considered to arrive at an optimum size which will give the least percent of error. The dimension of slab taken was 3.5m×4.5m and the foundation was modeled as surface springs on the bottom face of the element with the surface spring stiffness the same as the modulus of strengthened sub base. Elements of varying sizes in plan from 100cm×100cm, to 20cm×20cm were taken. The critical stresses for the various element sizes were analyzed. The results are shown in Figure 4.4. The optimum element size was found to be 25cm×25cm when the error is minimized to about 4%.

A schematic diagram of the slab showing the edge restraints is given in Figure 4.5. Some of the salient features of the FEM model are:

a) The elements taken were of size 25cm X 25cm X 33cm. The maximum stresses developed were in agreement with the analytical formulae as given by modified Westergaard's Equation 2.2 in Chapter 2.

IRC tensile stress (9.1 kg/sq cm)

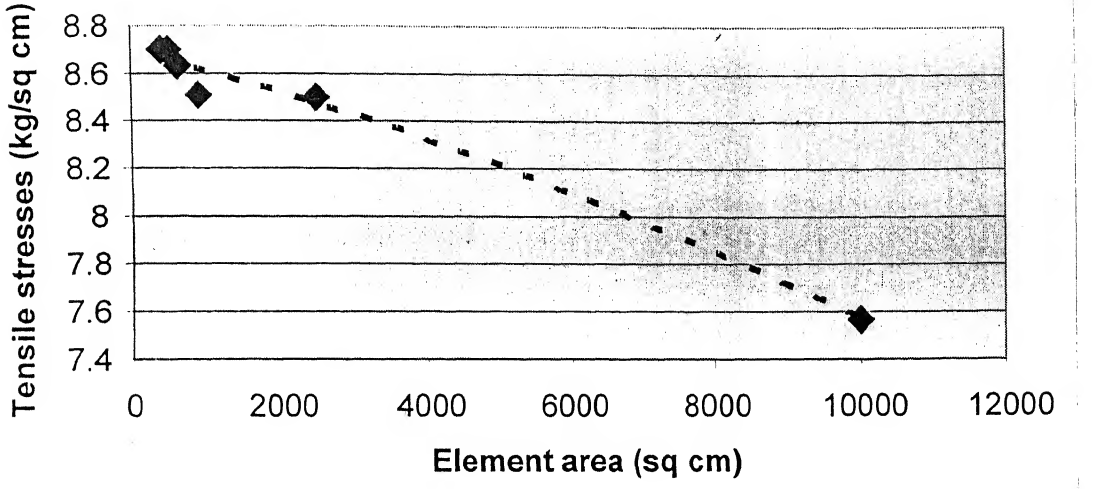


Figure 4.4: Optimization of element size

b) As seen from the Figure 4.5 the dowelled joint allows for movement in the lateral direction and rotation about the transverse joint. Hence both the dowelled transverse joints have been modeled with lateral movement and rotation about the transverse joint being allowed.

c) The axle loads irrespective of the single wheel/dual wheel or single axle/tandem axle nature, have been converted into equivalent circular area based on the tyre pressure. Since the program does not provide for elliptical or circular elements the loading has been applied using elements of rectangular shape. However the area of loading has been kept the same to the extent possible.

d) The loading was carried out at the longitudinal edge and then shifted towards the interior of the slab to gauge the variation in stresses experienced for the models and the point of maximum stress.

e) The FEM modeling was carried out for a dimension 3.5m X 4.5m for the normal and stiffened slab. The shoulder was included in another model and the dimension here was 4.5m X 4.5m.

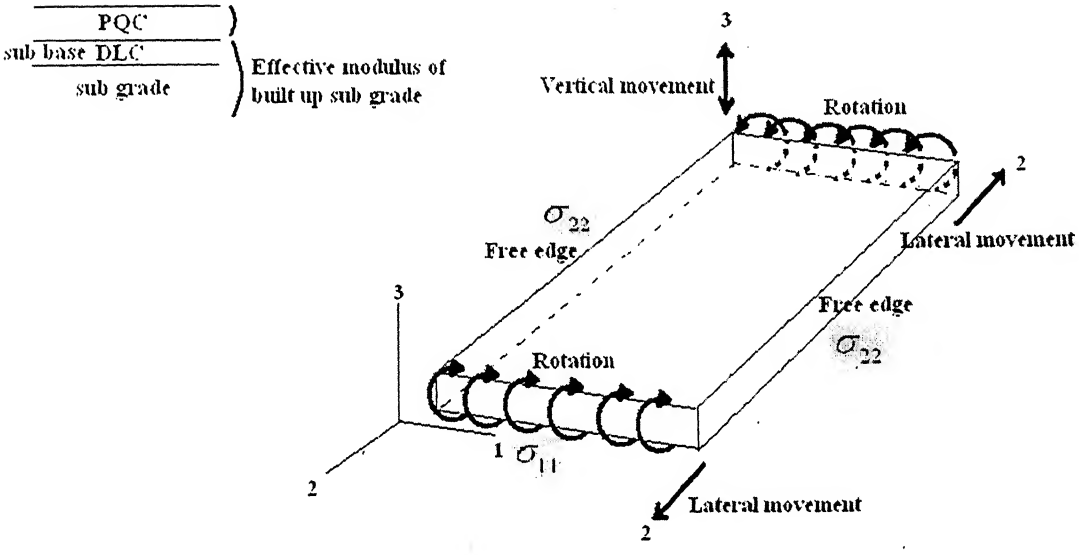


Figure 4.5: Schematic diagram showing the edge restraints and governing stresses

4.3 Illustrative example

The input data used is based on the actual data obtained from an ongoing NHAI project. The details of the project and the cross section of the pavement under construction are given in Appendix 'A'. The salient features of the input data are summarized below.

a) Modulus of sub grade:

i. Modulus of built up sub grade with DLC 10.60 kg/cm^3

	ii.	Thickness of DLC	150mm
b)	Properties of cement concrete (used in PQC)		
	i.	Design flexural strength of concrete (90 days)	45 kg/cm^2
	ii.	Modulus of elasticity of concrete	3.0×10^5 kg/cm^2
	iii.	Poisson's ratio	0.15
	iv.	Coefficient of thermal expansion	10×10^{-6} / °C
c)	Traffic factors		
	i.	Number of commercial vehicles (Both directions)	5509
	ii.	Traffic growth rate per annum	6.5 per cent
	iii.	Tyre pressure	7.0 kg/cm^2
d)	Load safety factor		1.2
e)	No: of lanes		2
f)	Design period		34 years
g)	No; of trucks per day (2 axles, multi axle trucks)		87%

h) Design traffic: The cumulative traffic over the design period is given by

$$C = \frac{365 \times A\left\{(1+r)^n - 1\right\}}{r} = 75{,}773{,}780 \text{ axles}$$

i) The axle load distribution is given in the Table 4.1. The number of repetition of axles with an average of two axles per vehicle is given

Table 4.1: Axle load distribution

Axle load class	Per cent	Repetitions
20-22	0.1007	1,52,610
18-20	0.0503	76,230
16-18	0.3020	4,57,670
14-16	0.6039	9,15,200
<14	98.9431	149.95×10^6

4.4 Stresses generated

In principle as the wheel load acts on the top of the slab, with the bottom being continuously supported, the top experiences flexural compressive stresses and the bottom, flexural tensile stresses. Now, given the brittle nature of concrete and its inability to take large tensile stresses, the critical stresses are the tensile stresses which are experienced at the bottom of the slab. The maximum compressive stresses are experienced on the face loaded, at the point of loading. The maximum tensile stresses experienced are in the face opposite, at a point directly below the point of loading. The stress contours shown in Figure 4.6 show that the magnitude of the stresses reduces rapidly with distance from the point of application of load.

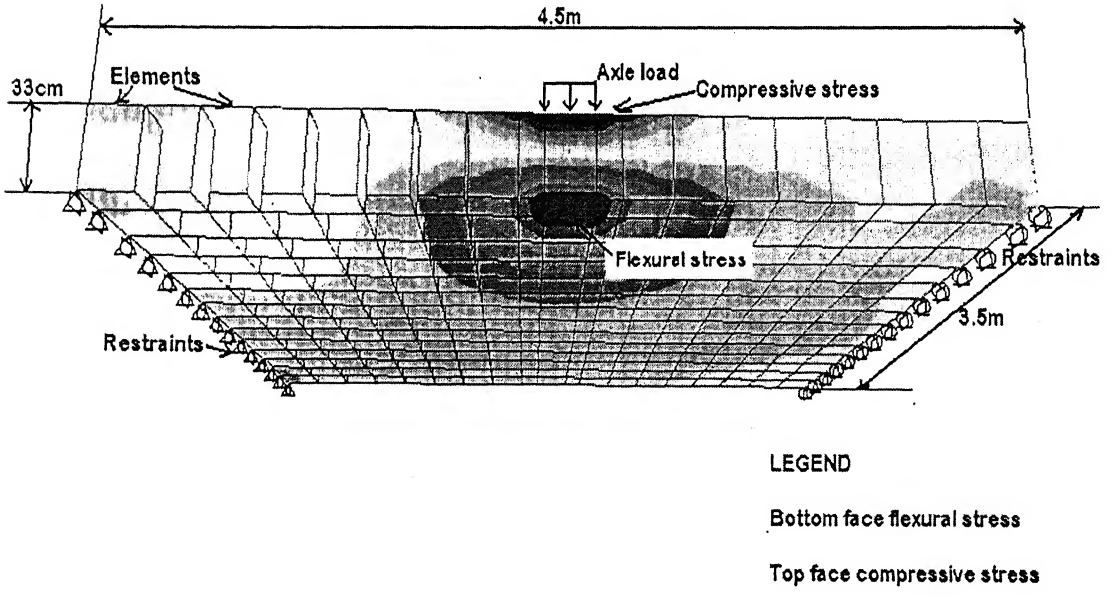


Figure 4.6 Stress contours on a loaded slab

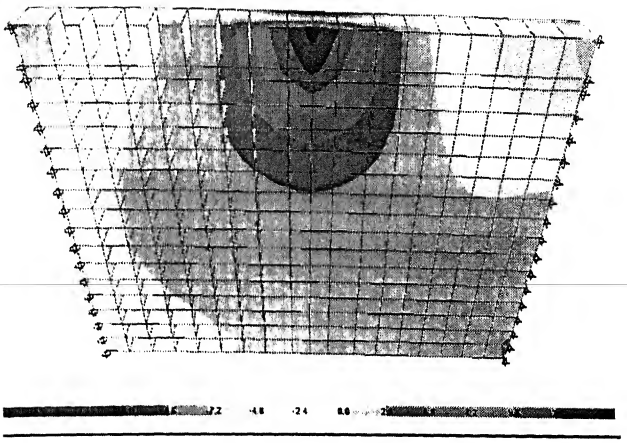
Thus the thickness of the pavement is based on the critical tensile stresses that develop midway of the longitudinal edge between two adjacent transverse joints.

4.4.1 Variation in critical stresses with location of loading and point of loading

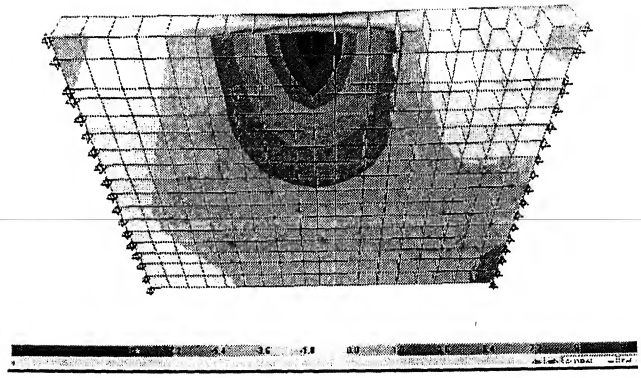
An axle load of 16800kg was applied at varying positions from the edge of the slab and the critical stresses experienced by the slab noted. It was found that as the load moves away from the edge the critical stresses experienced by the slab reduce. The variation was found to be of the order of about 50% of the edge stresses in this case. The stress contours of the tensile stresses experienced at the bottom of the slab at different points of application of the load from the edge is given in Figure 4.7. The results are tabulated in Table 4.2 and the variation of the stresses as the point of application of the load moves away from the edge given in Figure 4.8.

Table 4.2: Variation in stresses towards interior of slab

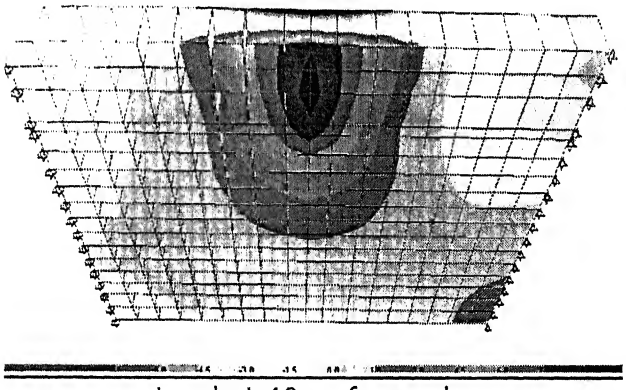
S No:	Position of wheel	Maximum flexural stresses (kg/cm^2)	
		Compressive	Tensile
1	On the edge	16.90	15.66
2	20 cm from the edge	13.22	12.27
3	40 cm from the edge	10.85	9.94
4	60 cm from the edge	9.60	8.89
5	At the center	8.87	7.99



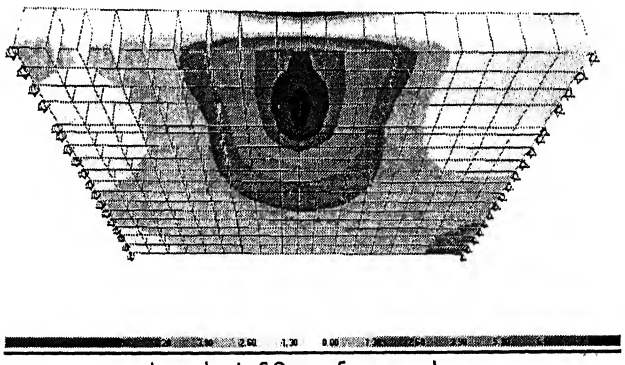
Load at the edge



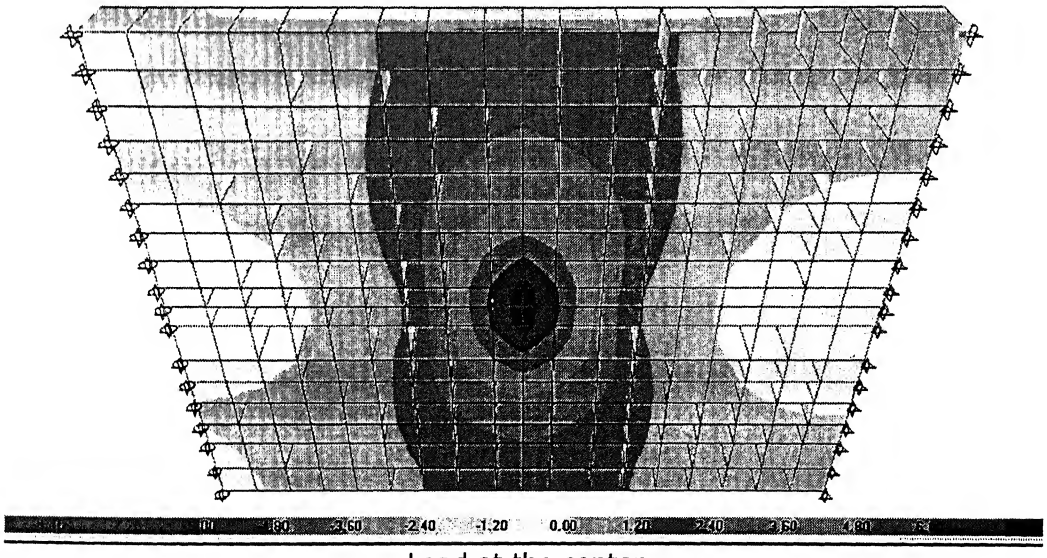
Load at 20cm from edge



Load at 40cm from edge



Load at 60cm from edge



Load at the center

Figure 4.7:Tensile stress contours at the bottom face of the slab for loads at different points from the edge

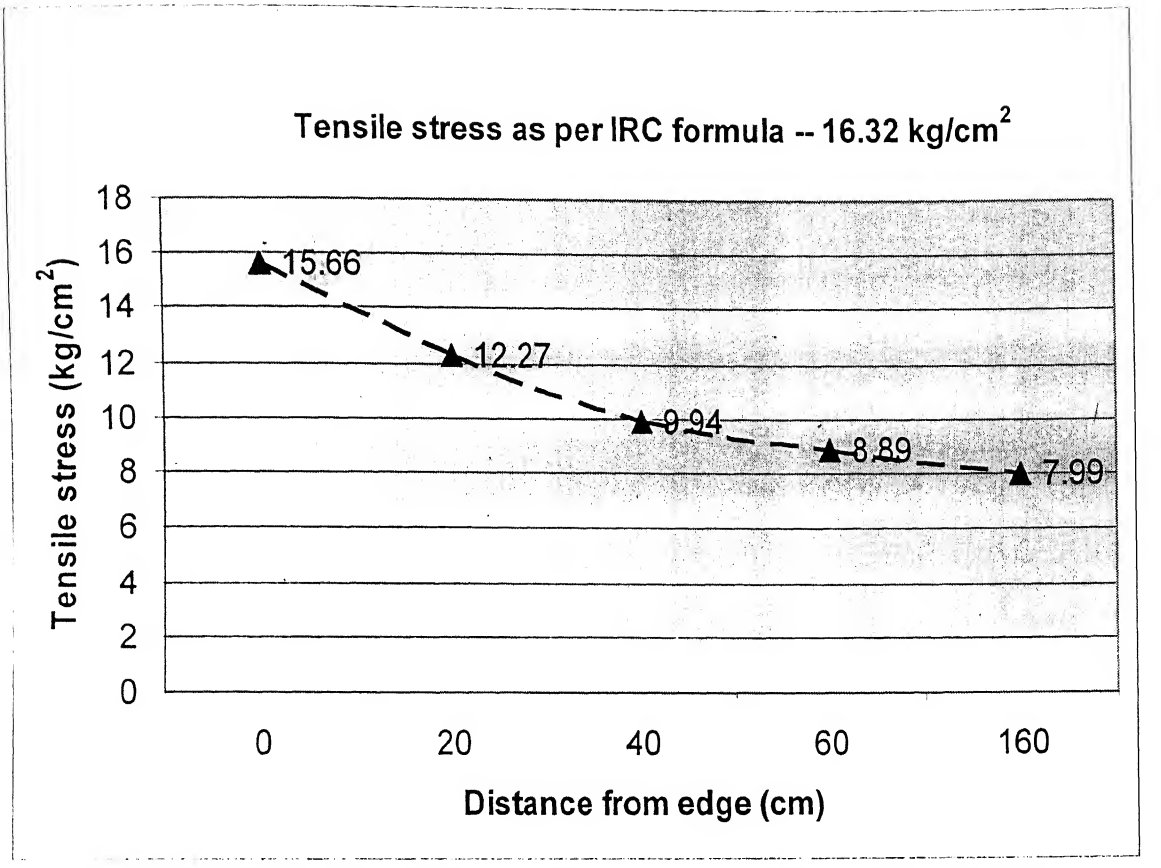


Figure 4.8: Reduction of tensile stresses with load moving from the edge

4.4.2 Standard rectangular slab

The standard slab of constant thickness (330mm) was analyzed using the SAP model. The input data used is given in section 4.3 above. The element size used was 20cm×25cm. Having determined the critical location of the load further calculation was carried out by varying the axle loads. The loading was at the edges, the critical stresses obtained are given in Table 4.3. The variation with the values obtained by using the Westergaard's formulae is given in the table in percentage. The analysis was carried out using the stresses from the IRC specifications. A slab thickness of 330mm gave a cumulative fatigue damage of 72.5% which is tabulated in Table 4.4. Since this value is less than 100% the slab design was found to be safe.

Table 4.3: Stresses of rectangular slab with uniform thickness

S No:	Wheel load (kg)	Stresses as per IRC (kg/cm^2)	Maximum flexural stress (kg/cm^2)		Percentage variation
			Compressive	Tensile	
1	8400	16.32	16.70	15.70	3.79
2	9600	18.30	18.53	17.06	6.77
3	10800	20.22	20.10	18.43	8.85
4	12000	22.08	21.71	20.23	8.37
5	13200	23.90	23.47	21.72	9.12

Table 4.4: Calculation for cumulative fatigue damage

Axle load	**AL X 1.2**	**Stress (kg/cm^2)**	**Stress ratio**	**Expected repetition (n)**	**Fatigue life (N)**	**Fatigue life consumed (n/N)**
7000	8400	16.32	0.36	149.95×10^6	Infinity	0
8000	9600	18.30	0.41	9,15,200	Infinity	0
9000	10800	20.22	0.45	4,57,670	Infinity	0
10000	12000	22.08	0.49	76,230	1,286,914	0.059
11000	13200	23.90	0.53	1,52,610	2,29,127	0.666

The total fatigue is 72.5% which is less than100%, hence safe.

Thus the specifications of the new rigid pavement is given underneath and the cross sectional view is given in Figure A1 of Appendix 'A'.

a) Thickness of the concrete slab 330mm

b) DLC thickness as the sub base 150mm

c) Drainage layer (gravel sand) 150mm

4.5 Effect of longitudinal stiffeners along ends on critical stresses

In the previous case the critical tensile stresses at the longitudinal edge was taken for designing the pavement slab. Thus the thickness provided through out the slab width was far higher than the actual requirement as the stresses reduce towards the interior as seen in section 4.4.1. Thus a more economical design would be one in which the slab is

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thinner with thicker edges which can take care of the higher stresses developed at the edges. A schematic representation of the slab is shown in Figure 4.9. The thickness of the edge is 35cm while the slab interior is 23cm. This configuration of a slab will lead to saving in high cost pavement quality concrete (PQC) which can be replaced by low cost dry lean concrete (DLC) sub base.

Here an analysis is carried out with the thickness of the slab as 23cm and stiffeners provided at the longitudinal edges. The stiffeners were 75cm wide and the slab thickness at this point was 35cm. To check the effect on the magnitude of stresses generated at the interiors due to reduction of the slab thickness the analysis was carried out by placing the different wheel loads at 25cm, 60cm and at the center in addition to the edge loading.

The tensile stress results obtained for different axle loads at various points of loading are tabulated in Table 4.5 and the stress contours for edge loading of the slab for the various wheel loads are shown in Figure 4.10.

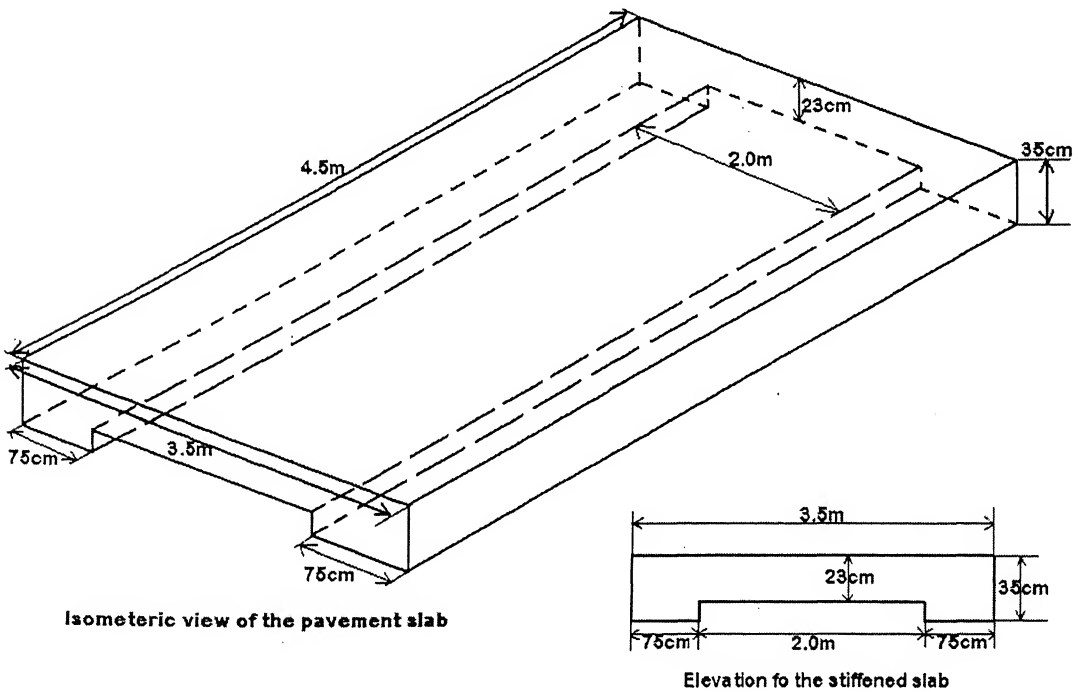


Figure 4.9: Plan and elevation of the stiffened slab

It can be seen that the critical tensile stresses experienced by the pavement were due to the edge loading in spite of reducing the thickness in the interior of the slab. The slight increase in the stresses experienced in the interior of the slab is due to the reduced thickness of the slab. The critical stresses from the edge loading were corrected using the variation arrived at in Table 4.3 and this corrected stress used in the fatigue analysis. The fatigue calculations are shown in Table 4.6 and the design was found to be safe.

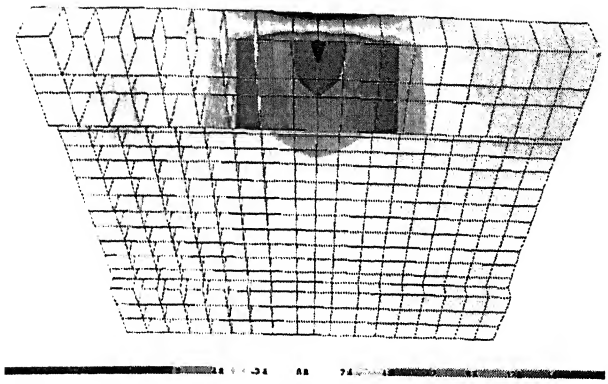
Table 4.5: Tensile stresses of stiffened slab for loading at various distances from the edge

S No:	Wheel load (kg)	Maximum tensile stress (kg/cm^2)			
		Edge	25cm from the edge	60cm from the edge	Center of slab
1	8400	15.17	13.97	10.97	13.32
2	9600	16.64	15.89	11.94	14.76
3	10800	18.72	17.88	13.20	16.26
4	12000	20.06	19.67	14.29	17.08
5	13200	21.79	20.59	15.39	18.69

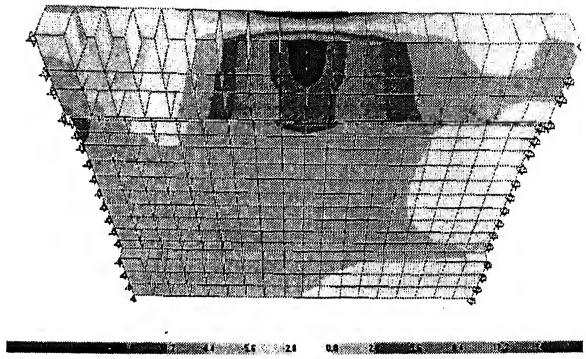
Table 4.6: Calculation for cumulative fatigue damage

Axle load	AL X 1.2	Edge Stress (kg/cm^2)	Corrected stress	Stress ratio	Expected repetition (n)	Fatigue life (N)	Fatigue life consumed (n/N)
7000	8400	15.17	15.78	0.35	149.95×10^6	Infinity	0
8000	9600	16.64	17.77	0.39	9,15,200	Infinity	0
9000	10800	18.72	20.38	0.45	4,57,670	Infinity	0
10000	12000	20.06	21.74	0.48	76,230	2,400,000	0.0317
11000	13200	21.79	23.78	0.53	1,52,610	2,29,127	0.666

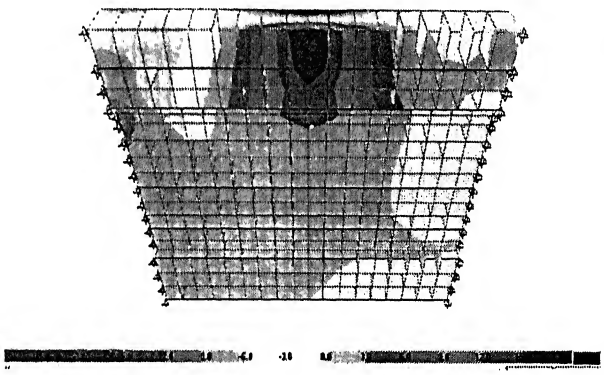
The total fatigue is 69.77% which is less than100%, hence safe.



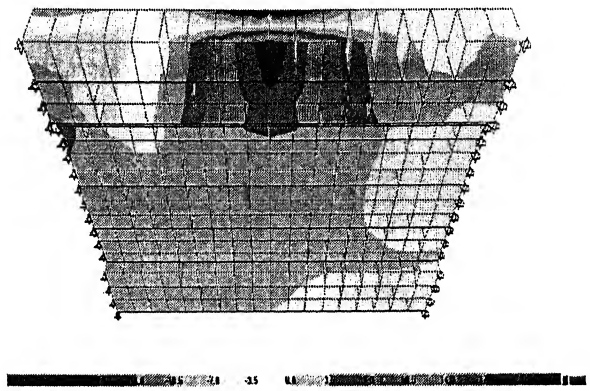
a) 8.4t edge wheel loading



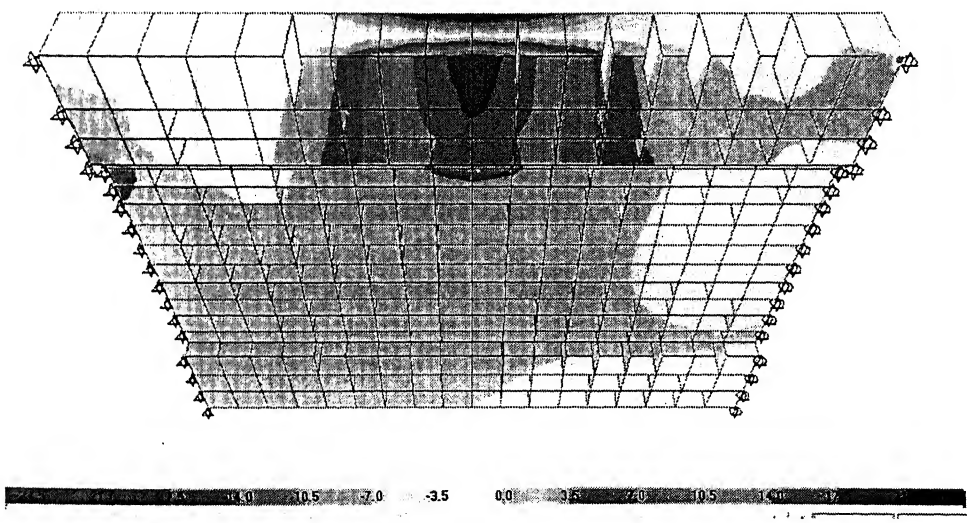
b) 9.6t edge wheel loading



c) 10.8t edge wheel loading



d) 12.0t edge wheel loading



e) 13.2t edge wheel loading

Figure 4.10 Stress contours in stiffened slab for various wheel loads

The specification of the pavement with longitudinal stiffeners is as under:

a) The thickness of the slab is 23cm with stiffeners of width 75cm and thickness 35cm being provided on both the longitudinal edges.

b) A DLC layer of width 200cm and thickness 12cm is laid to fill the gap between the stiffeners.

c) A DLC layer 15cm thick is provided as the sub base.

d) A drainage layer (gravel sand) 0f 15cm thickness is provided.

4.6 Effect of including the shoulders in the analysis for stresses in slabs

As shown in Figure 4.1 the outer traffic lane is provided with a shoulder, also made with PQC. The total width including the shoulder and the lane width is laid by the paver with the tie bars in place and after the completion of the initial curing time the longitudinal joints sawn thus providing a shoulder which is structurally separate from the pavement structure. Inclusion of the shoulder as an integral part of the pavement slab means that large part of the loading will be in the lane edge which is away from the slab edge as the shoulder is now an integral structural part of the slab. Thus the slab will experience low stresses consequently leading to an increase in the number of repetitions it can take before fatigue failure sets in. Also the traffic on the shoulder is mainly the encroaching traffic and the parking traffic which varies between 3.5% and 10% of the design traffic. Thus the number of repetitions of critical loading will also be lower leading to a more economical design. However in the analysis carried out the original design traffic was used since data on the likely traffic on shoulders is not available at present.

The slab dimensions taken were 4.5m×4.5m which was inclusive of the shoulder of 1m to be provided. The mesh size taken here was 25cm×25cm. The thickness of the slab taken was 23cm with stiffeners provided at the longitudinal edges of width 75cm and

thickness 35cm. The plan and elevation of the slab is given in Figure 4.11. The loading at various points towards the center of the slab was analyzed to find the variation

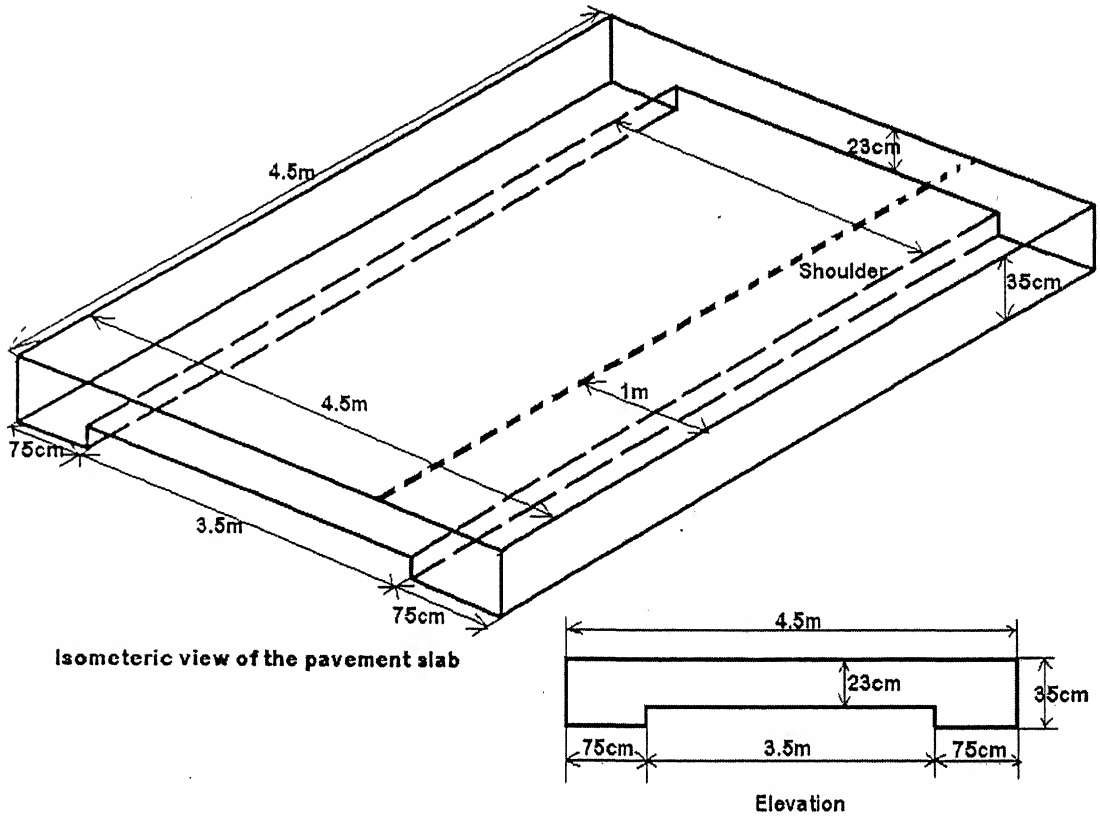


Figure 4.11: Plan and elevation of the slab with shoulder and stiffened edges

of the stresses with change in point of application of the load. The critical stresses were found to be for the edge condition. However the stresses towards the center where found to be higher though well below the stresses experienced due to edge loading. The stress contour on the top and bottom of the slab as the point of application of the load moves away from the edge is given in Figure 4.12. This trend indicates that as the dimensions x and y directions are equal the lack of stiffener in the other two sides is affecting the behavior of the slabs under loading. The stresses got for the various loading conditions

are tabulated in Table 4.7 and the fatigue calculations given in Table 4.8. It is found that the design thickness taken is safe.

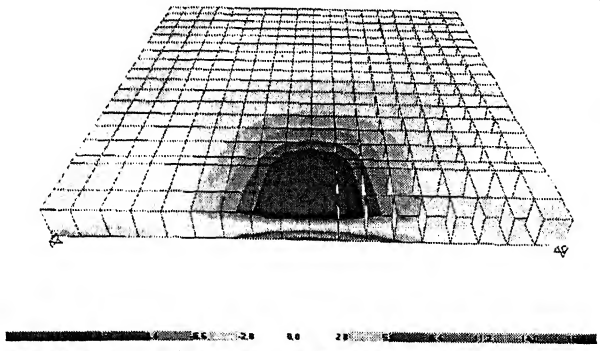
Table 4.7: Stresses of stiffened slab with shoulder

S No:	Wheel load (kg)	Maximum flexural stress (kg/cm^2)			
		Edge	25cm from the edge	75cm from the edge	Center of slab
1	8400	15.19	12.93	9.50	13.20
2	9600	17.37	14.64	10.63	15.05
3	10800	18.50	16.10	11.66	16.09
4	12000	20.04	17.30	12.62	17.01
5	13200	21.97	18.84	13.63	18.60

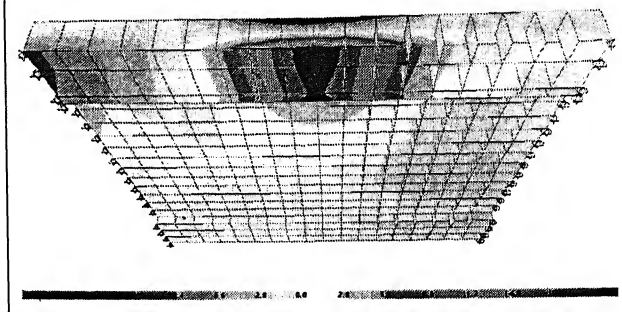
Table 4.8: Calculation for cumulative fatigue damage

Axle load	AL X 1.2	Stress (kg/cm^2)	Corrected stresses	Stress ratio	Expected repetition (n)	Fatigue life (N)	Fatigue life consumed (n/N)
7000	8400	15.19	15.77	0.35	149.95×10^6	Infinity	0
8000	9600	17.37	18.55	0.39	9,15,200	Infinity	0
9000	10800	18.50	20.14	0.41	4,57,670	Infinity	0
10000	12000	20.04	21.72	0.483	76,230	1,966,925	0.0388
11000	13200	21.97	23.97	0.533	1,52,610	207,523	0.7354

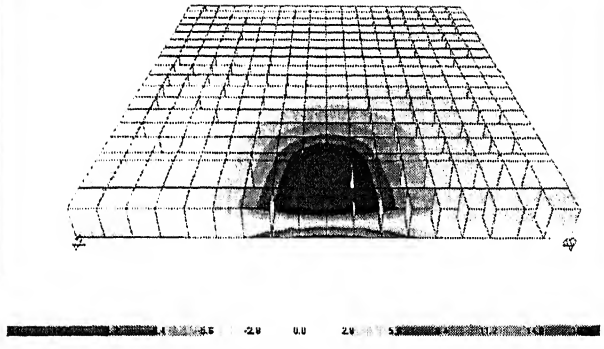
The total fatigue is 77.42% which is less than100%, hence safe.



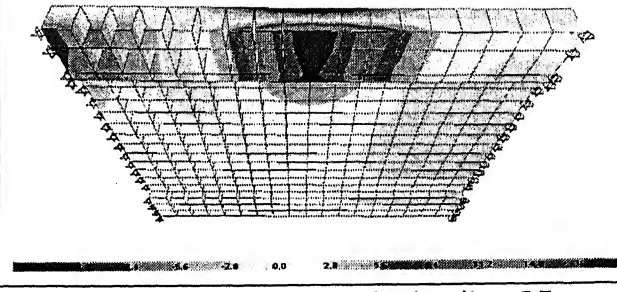
a) Compressive stress contours for edge loading



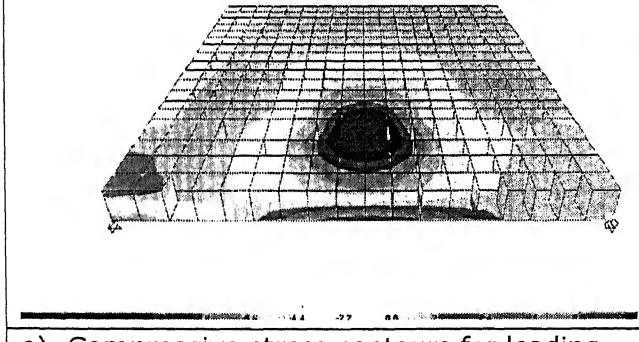
b) Tensile stress contours for edge loading



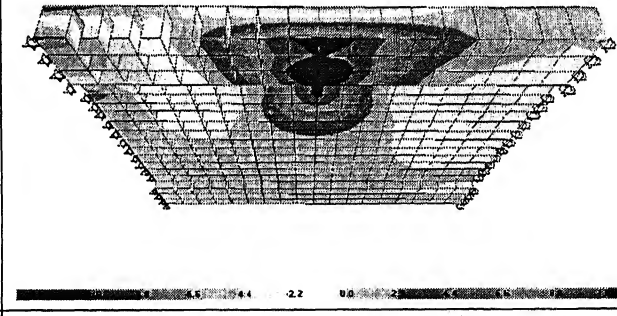
c) Compressive stress contours for loading 25cm from edge



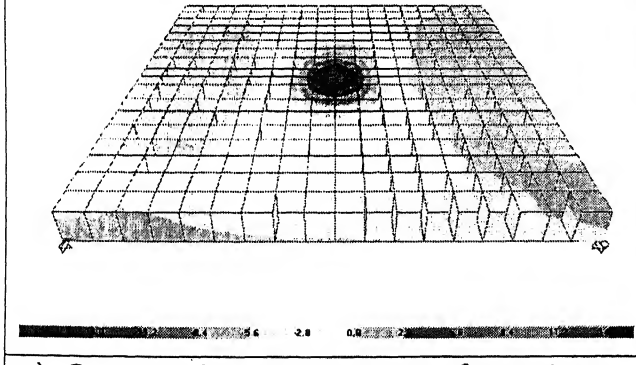
d) Tensile stress contours for loading 25cm from edge



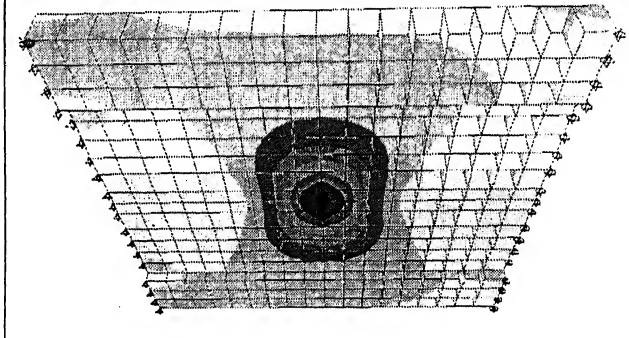
e) Compressive stress contours for loading 75cm from edge



f) Tensile stress contours for loading 75cm from edge



g) Compressive stress contours for center loading



a) Tensile stress contours for center loading

Figure 4.12: Stress contours on the top and bottom face of slab for wheel load of 13.2t

Thus the specification of the slab with stiffened edges and integral shoulder is as under:

a) The thickness of the slab is 23cm and of dimension 4.5m×4.5m which includes the shoulder of 1m

b) Stiffener of width 75cm and thickness 35cm is provided on both the longitudinal edges.

c) A DLC layer of width 300cm and thickness 12cm is laid to fill the gap between the stiffeners.

d) A DLC layer 15cm thick is provided as the sub base.

e) A drainage layer (gravel sand) 0f 15cm thickness is provided.

4.7 Transverse edge loading

Since the slab thickness was reduced at the transverse joints, an analysis was carried out to check the magnitude of stresses which are developed for the various axle loads. The loading was done at the edge, mid point from the two longitudinal joints. The analysis was carried out for both the models considered earlier. The transverse edge restraints were changed to find the stresses that will develop due to various situations. The conditions considered were

a) When the lateral movement and rotation about the transverse joint is permitted. This represents the normal case where the dowelled joint is performing as desired.

b) When lateral and vertical movement along with rotation about the transverse joint is permitted. This represents a condition when there is a partial failure in the effectiveness of the dowelled joint.

c) When the joint is modeled as completely free i.e., when all restraints are released. This represents a complete failure of the joint.

The results are tabulated in Table 4.9. It is seen that the stresses developed are lower than the maximum stresses developed due to longitudinal loading in cases (a) and (b). However when the joints were modeled as free the maximum stresses developed increased steeply. On comparison it was found that these stresses were same as the stresses which would have been experienced if a slab of thickness 23cm been loaded on the mid of the longitudinal edge.

Table 4.9: Stresses developed due to transverse edge loading

S No:	Wheel load (kg)	Maximum stresses on longitudinal edge (kg/cm^2)	Maximum tensile stress (kg/cm^2)					
			Restraints y and M_x free		Restraints y, z and M_x free		Edge free	
			σ_{11}	σ_{22}	σ_{11}	σ_{22}	σ_{11}	σ_{22}
Stresses developed on transverse loading in stiffened slab								
1	8400	15.17	7.24	10.22	14.28	5.5	28.91	7.59
2	9600	16.64	8.50	11.98	14.56	6.05	31.90	7.84
3	10800	18.72	9.12	13.46	15.96	6.85	34.65	8.84
4	12000	20.06	10.49	14.71	17.35	7.32	37.07	9.30
5	13200	21.79	11.04	16.01	19.01	8.01	39.34	10.16
Stresses developed on transverse loading in stiffened slab with integral shoulder								
1	8400	15.19	6.97	10.14	12.32	4.79	28.66	7.53
2	9600	17.37	8.25	12.07	13.97	5.06	31.73	7.88
3	10800	18.50	8.83	13.56	14.78	5.68	34.49	8.87
4	12000	20.04	9.29	14.84	15.38	6.25	36.71	9.75
5	13200	21.97	10.62	16.16	16.71	6.40	39.24	10.18

4.8 Temperature stresses

The temperature differential experienced by the top and bottom surface of the slab reduces with reduction in thickness and thus the temperature stresses are less at the portions where the thickness is reduced. Thus the stresses which are worked out from the specification are safe. In practice the stresses experienced due to temperature will be less at the portion where the slab is thin and increases on increase of thickness of the slab. The maximum temperature that the slab will experience can be as per Westergaard's analysis using Bradbury's coefficient.

$$S_{te} = \frac{E\alpha tC}{2} \tag{4.1}$$

where

S_{te} temperature stress in the edge region kg/cm^2

E modulus of elasticity of concrete kg/cm^2

α coefficient of thermal expansion of cement concrete /^{o}C.

t maximum temperature differential between top and bottom of slab during day /^{o}C.

C Bradbury's coefficient which can be directly ascertained from the chart using L/l and B/l values where l is the radius of relative stiffness found from the following formula

$$l = \sqrt[4]{\frac{Eh^3}{12(1-\mu^2)k}} \text{ where} \tag{4.2}$$

h thickness of the slab

μ poisson's ratio

k modulus of sub grade reaction kg/cm^3

Substituting the values we have:

l = 100.85 cm

L/l = 450/100.85 = 4.462. The corresponding value of C is 0.6136. The temperature differential is 21 and we get the maximum temperature to be 15.0946 kg/cm^2

The total edge stress is 23.97+15.0946 = 39.0646 kg/cm^2 which is less than the flexural stress of concrete (45 kg/cm^2) and hence safe.

4.9 Corner stresses

The corner stresses are dependent on the radius of relative stiffness and the thickness of the slab at the corner. In all the cases considering that the thickness given is 35 cm we have,

$$\text{Corner stress} = \frac{3P}{h^2}\left[1-\left(\frac{a\sqrt{2}}{l}\right)^{1.2}\right] \quad \text{where} \tag{4.3}$$

l relative radius of stiffness found out earlier 100.851 cm

a area of contact of the wheel, considering a single axle dual wheel

we have

$$a = \left[0.8521\times\frac{P}{q\times\pi}+\frac{S}{\pi}\left(\frac{P}{0.5227\times q}\right)^{0.5}\right]^{0.5} \quad \text{where} \tag{4.4}$$

P wheel load (13200 kg)

q tyre pressure (7 kg/cm^2)

S c/c distance between the tyres (31cm)

Substituting the values we get a = 24.51 cm

The corner stress is 23.355 kg/cm^2 which is less than the flexural stress of concrete 45 kg/cm^2 and hence safe. An analysis of the slab by SAP shows that the corner stresses for stiffened slab and the shoulder integral stiffened slab are 22.45 kg/cm^2 and 27.8 kg/cm^2 respectively, both of which are less than 45 kg/cm^2 and hence safe. However it is pertinent to note that the stresses are experienced only when the edges are free which not the normal case as the transverse joint is dowelled and thus the stresses experienced are less than the critical stresses in the longitudinal stresses.

4.10 Advantages of the alternative designs

Both the designs suggested involve in provision of stiffeners along the edges of the slab. Though there was an increase in the stresses experienced in the interior portion of the slab mainly due to the reduced thickness, the governing stress was still that due to edge loading. The major advantage is the saving in the material used for pavement construction. The volume of PQC saved per kilometer in both cases as given and tabulated in Table 4.10

a) Providing and laying PQC per m^3 = Rs 3983.17

b) Providing and laying DLC per m^3 = Rs 1556.79

c) Total volume of PQC required for a kilometer of pavement of width 3.5m = $1000\times3.5\times0.33 = 1155m^3$.

d) Total volume of PQC required for a kilometer of pavement of width 4.5m = $1000\times4.5\times0.33 = 1485m^3$.

e) Total volume of PQC required for the slab with stiffener = $[1000\times3.5\times0.35]-[1000\times2\times0.12] = 985\ m^3$.

f) Total volume of PQC required for the slab including shoulders and stiffened edges = $[1000\times4.5\times0.35]-[1000\times3\times0.12] = 1215\ m^3$

g) Saving in PQC

   i. Slab with stiffeners: $1155-985 = 170m^3$. Volume of DLC required is $240\ m^3$.

   ii. Slab including shoulders and stiffened edges: $1485-1215 = 270\ m^3$. Volume of DLC required is $360\ m^3$.

f) Savings in cost per kilometer of single lane pavement.

   i. Slab with stiffeners: $[170\times3983.17]-[240\times1556.79] = 3,03,509.30$

ii. Slab including shoulders and stiffened edges: [270×3983.17]-[360×1556.79] = 5,15,011.50

Table 4.10: Economy of the models

Model	Volume of PQC (normal slab) (m^3)	Volume of PQC (stiffened slab) (m^3)	Volume of PQC (shoulder integral stiffened slab) (m^3)	Saving in volume of PQC (m^3)	Saving in cost (Rs)
Normal slab	1155	985	–	240	3,03,509.30
Shoulder integral slab	1485	–	1215	360	5,15,011.50

In the second case the shoulder being an integral part of the pavement ensures that the flexural stresses experienced by the slab due to vehicle loading on or at the edge of the lane is low. Thus the slab will be able to take on large number repetitions. Also the traffic loads on the shoulder is basically consists of only encroaching traffic and parking traffic which is generally less than 10% of the total cumulative design traffic. Thus the design traffic for which the slab is to be designed will be far lower than in the case of a normal slab.

4.11 Summary

An analysis of two models of variable thickness pavement design where the thickness is varied commensurate to the reduction in critical stresses on interior loading was considered. The efficacy of the design was established by carrying out a comparative analysis of the models with that of the standard design where constant thickness of pavement is used. Some of the facts brought out by the study are:

a) Economical design of balanced pavements based on varying thickness is feasible.

j) In the design where the shoulder was considered to be structurally integral to the main slab, the design traffic taken was the same as that of the normal pavement as study on the nature of encroaching traffic and parking traffic in Indian highways is not available. However the standard quantity taken for encroaching traffic and parking traffic is 3.5% and 0.8% respectively. Even taking into consideration he likely traffic diversion for maintenance etc, the net traffic is likely to be in the range of 10% of the design traffic. Thus the actual design traffic in this case will be less enabling a far more economical design.

However the effect of variable settlement, loss of support and other likely problems which might be encountered require further studies so as to fine tune the design and these are discussed in the conclusion being covered in the next chapter.

Chapter 5

Concluding remarks

5.1 General

This thesis focused on the study of Indian specifications for plain concrete pavement construction. A comparative analysis of specifications published in various countries was carried out and specific areas in which further research is needed make the Indian design more economical and relevant to the local conditions were identified. Analyses of a few innovative models were carried out and the likely savings in cost that can be incurred through such design worked out. The conclusions of the work are given in this chapter, along with the limitations of the work, and some further work that could be taken up.

5.2 Summary of findings

A comparative study thicknesses arrived at from the design carried out in accordance with various specifications shows that the Indian specifications result in a thickness that is 10-14% higher than that by others. Reducing a centimeter of thickness in a 7m wide road, could result in a saving of Rs 2.4 lakhs per kilometer of pavement, and several thousands of kilometers of pavement are in various stages of planning, it is very important that a closer look at the design parameters and specifications is taken at the earliest.

It is also seen that the some of the other specifications, such as AASHTO, are more comprehensive and rational in nature, and take into consideration the effect of various factors, such as, loss of support to the pavement due to erosion of sub base, environmental factors, provision of shoulders, load transfer due to provision of dowels

and aggregate interlock, etc. on the performance of the pavement. Given the diversity in conditions in terms of soils, temperatures, etc. it is necessary that some of these areas are incorporated into our design philosophy and methodology, to ensure a more economical and relevant design.

The pavement is designed for stresses which are experienced by the slab due to edge loading. From the analysis carried out in this study, it was found that as the (same) load moves towards the interior of the slab, the stresses reduce drastically. A different and more economical cross-section of the pavement has been suggested in this study, though further work to study the variation in stresses at corners, etc. needs to be carried out along with field investigations and trials. Also the present concept of providing tied shoulders should be critically examined as the stresses experienced by the slab due to loads on the lane edge can be reduced if the shoulder is made integral to the main slab and designed as such. Moreover the load repetition on the shoulder would be due to the encroaching traffic and the parking traffic which is generally taken to be less than 5-10% of the design traffic. Thus the fatigue damage will be far less if the shoulder is made an integral part of the main slab.

5.3 Applications of present work

A revision of the Indian specification based on the research carried out in areas specified in the section above will make the design more economical and relevant to the local situations. The feasibility of adoption of the balanced model where the thickness of the pavement can be varied in accordance with the reduction in stresses experienced and the possibility of structurally integrating the shoulder to the main slab established analytically. The alternative models suggested will enable a savings in the cost to the tune

of 7-9%. The costlier PQC material can be replaced by an extra layer of DLC which provides good stiffness and is cheaper.

5.4 Limitations and suggestions for future work

The affect of various factors on the performance of the pavement cannot be predicted mechanistically. The specifications published by various countries depend on empirical relations arrived at from data collected over a period of time by monitoring the performance of full scale experimental models. Such an approach is required to make the Indian specification more relevant to local conditions and economical. Also, the feasibility of a pavement model with varying thickness has been established, however further detailed studies are required before such economical models can be adopted for execution on ground. Some of the areas which require further studies for an in-depth understanding of the various aspects which affect the functioning of the concrete pavement are brought out in the subsequent sub sections.

5.4.1 Strengthening Indian specifications

Further studies in the following areas will better correlate the various parameters affecting the pavement performance, thus making the Indian design economical and more sensitive to the Indian terrain and working conditions.

a) **Erosion distress** Loading of the pavement by tandem and multi axle loads causes deflection of the pavement leading to pumping and erosion of the sub base which causes loss of support. Repetitive loading of this nature leads to distress for which the pavement is to be analyzed. Erosion analysis is not catered in the Indian design as at present the percent of traffic consisting tandem and multi axle load is small. How ever the point to be

noted is that the concrete pavements are designed for a period of 20-30 years when the mix of traffic will change vastly and hence this aspect should be taken into consideration in the design.

b) **Reliability factor** The Indian design caters for a safety factor which is factored into the axle load to cater for the overloading of trucks which is rampant on our roads. Depending on the likely density of truck traffic on different types of roads the safety factor is quantified. However a more realistic approach will be to consider a case where the error in traffic prediction, traffic studies, etc., variation in analysis of the pavement model, importance of the road, nature of the road users etc, is taken into consideration to arrive at the range of safety factors.

c) **Provision of concrete shoulders** One of the major forms of distress which occurs in pavements is the erosion of the sub base. The cyclic loading, presence of moisture, deterioration of the materials used in sub grade/ sub base lead to a situation where the modulus of the sub base reduces leading to the pavement experiencing higher stresses on loading. Thus the fatigue damage sets in earlier leading to the onset of cracking. The best method of prevention of this is to provide for shoulders which would provide a support to the main slab, reduce the pumping and deflection due to reduction of edge and corner deflection, and prevent erosion of sub base as the water infiltration reduces as also the water is drained of away from the traffic lane. The Indian design recommends the provision of tied concrete shoulders but does not cater for the economy in design thickness this will accrue. Studies need to be carried out to incorporate this aspect in the design.

d) **Drainage** The presence of moisture has a detrimental effect on the modulus of the sub grade. Water leads to pumping and degradation of paving materials and provision of drainage layer prevents the ingress of moisture into the pavement system and this will have an effect on the pavement design. Indian specification provides for a drainage layer depending on the high flood level, though the effect of the same is not considered in the design.

e) **Environmental effect** The amount of rainfall, the type of soil, the nature of terrain, the climate etc, have a major impact on the design of the pavement. Adequate study is to be carried out to understand the inter relation between various factors such as moisture, temperature, soil swelling, frost heave, etc which affect the performance of pavement and their variation with the change in terrain and climate across the country. These should be incorporated to enable analysis and design of pavement suiting the local conditions and requirements.

5.4.2 Validation of alternative models

The following are the likely limitations of the proposed alternative models of the pavement slab design which should be further analyzed.

a) Here a sample problem has been taken and analyzed to bring out the feasibility of designing balanced pavements with varying thickness. Further analysis is required to standardize the design over a wide range of values of modulus of sub grade reaction (k), varying thickness of slab and stiffeners.

b) Water and moisture causes the erosion of sub base due to water infiltration as also due to water which is drained onto the shoulder edge. This will lead

to likely distress of the shoulder due to loss of support. This is likely to affect the main slab of which the shoulder is now an integral part. However this has to be seen in the light of the reduced critical load repetitions the pavement will be subjected to as the design traffic now is vastly reduced since only the encroaching traffic and parking traffic which is 3.5% and 0.8% respectively of the original design traffic is to be catered for. Even taking into consideration the likely traffic diversion for maintenance etc, the net traffic will be in the range of 10% of the design traffic. Thus this will offset the likely disadvantages due to loss of support. This aspect requires further study on the basis of a full scale experimental model and studies on the likely traffic which the shoulder takes in the Indian scenario.

c) The use of DLC as a sub grade provides for very high modulus of sub grade reaction. Also the provision of stiffeners may lead to better bonding of the sub grade to the pavement. Thus an analysis of the combination as a composite slab will lead to further economy of design. This aspect needs to be further researched upon.

d) As the design of the transverse joint is critical in the event of the failure of the joint due to the reduced thickness, further research will have to be carried out to develop efficient joints.

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Appendix A

Project details of concrete pavement under execution by NHAI

1. Interestingly for some of the projects being executed by the National Highway Authority of India in India, the guidelines followed by the Consultants are closer to that of the Portland Cement Association, and their charts, etc. have been used in the design of pavement thickness. Some of the salient features of the design are briefly discussed below along with a comparison with the relevant IRC provisions.

a) The design performance period taken was 30 years and design traffic period as 4 years, thus leading to a net design period of 34 years. The Indian design the design period advocated is 30 years which is the normal life span of a concrete road while 20 years is to be taken in cases where traffic cannot be predicted accurately and for low volume roads.

b) The stress analysis used is that of Westergaard's method which is the same as in IRC-58:2002.

c) The flexural strength of concrete is taken as 40 kg/cm^2 at 28 days while for pavement design purposes the 90 day strength of 45 kg/cm^2 was taken.

d) It is considered that the concrete can take infinite number of repetitions if the stress ratio is less than 50%. As per IRC the infinite loading is considered when the stress ratio is less than 45%.

e) The wheel load is increased by a factor of 20% to cater for possible increase of load due to impact and as a safety factor. In IRC the increase catered for is the same but is to take care of rampant overloading prevalent on our roads.

f) The traffic is categorized into axle groups and the stress for each axle group determined for further analysis. Here the two and multi axle vehicles are averaged into two axle repetition per vehicle.

g) The design CBR of the sub grade is taken as 6 per cent and modulus of sub grade soil reaction is 4.50 kg/cm^3. A 150 mm of DLC is provided over the sub grade as sub base and the modulus of the strengthened sub base taken as 10.6 kg/cm^3. While as per IRC the effective k value of a sub base of 150mm DLC over a sub grade with a k value of 4.50 kg/cm^3 is given to be 24.25 kg/cm^3.

The radius of contact is assumed to be circular and is based on the formulae $a = \sqrt{\frac{P}{\pi \times p}}$ where P is the wheel load and p is the tyre pressure. The same is considered even in case of dual wheels. The IRC advocates the use of $a = \left[0.8521 \times \frac{P}{q \times \pi} + \frac{S}{\pi} \left(\frac{P}{0.5227 \times q} \right)^{0.5} \right]^{0.5}$ where P is the wheel load, q tyre pressure and S is the c/c distance between the tyres which is taken as 31cm. This gives a lower value of radius of contact which results in slightly lower values of stresses.

2. The input parameters used for the design are given underneath.

a) Modulus of sub grade:

i.	Design CBR of sub grade soil	6 per cent
ii.	Modulus of sub grade soil reaction	4.50 kg/cm^3
iii.	Thickness of DLC	150mm
iv.	Modulus of built up sub grade with DLC	10.60 kg/cm^3

b) Properties of cement concrete

j)	Design flexural strength of concrete (28 days)	40 kg/cm^2
k)	Design flexural strength of concrete	45 kg/cm^2

(90 days)

l) Modulus of elasticity of concrete 3.0×10^5 kg/cm^2

m) Poisson's ratio 0.15

n) Coefficient of thermal expansion 10×10^{-6} / °C

c) Traffic factors

i. Number of commercial vehicles (both directions) 5509

ii. Traffic growth rate per annum 6.5 per cent

iii. Tyre pressure 7.0 kg/cm^2

d) Load safety factor 1.2

e) No: of lanes 2

f) Factor for directional distribution 0.50

g) Factor for lane distribution 0.75

h) Design period 34 years

i) No; of trucks per day (2 axles, multi axle trucks) 87%

j) Design traffic: The cumulative traffic over the design period is given by

$$C = \frac{365 \times A\left\{(1+r)^n - 1\right\}}{r} = \frac{365 \times 4792\left\{(1+0.065)^{34} - 1\right\}}{0.065} \times 0.50 \times 0.75$$

= 75,773,780 axles

j) The axle load distribution and the load repetition over the design period is given in the Table A1. The number of repetition of axles with an average of two axles per vehicle is given in the Table. (e.g. in the 20-22 class, the repetitions are 0.1007*75.8*2 = ___)

Table A1: Axle load distribution

Axle load class	Per cent	Repetitions
20-22	0.1007	1,52,610
18-20	0.0503	76,230
16-18	0.3020	4,57,670
14-16	0.6039	9,15,200
<14	98.9431	149.95×10^6

3. As per the design analysis a thickness of 330mm was got when the cumulative fatigue damage was 64% which was less than 100% and hence safe. A check for the curling stresses and the corner stress was carried out and found safe. The design cross section of the pavement which is being executed on the ground is.

a) Thickness of the concrete slab 330mm

b) DLC thickness as the sub base 150mm

c) Drainage layer (gravel sand) 150mm

4. The cross section of the rigid pavement is given in figure A1, the layout of the dowel and tie bars given in figure A2 and the details of the dowel and tie bar placement given in figure A3.

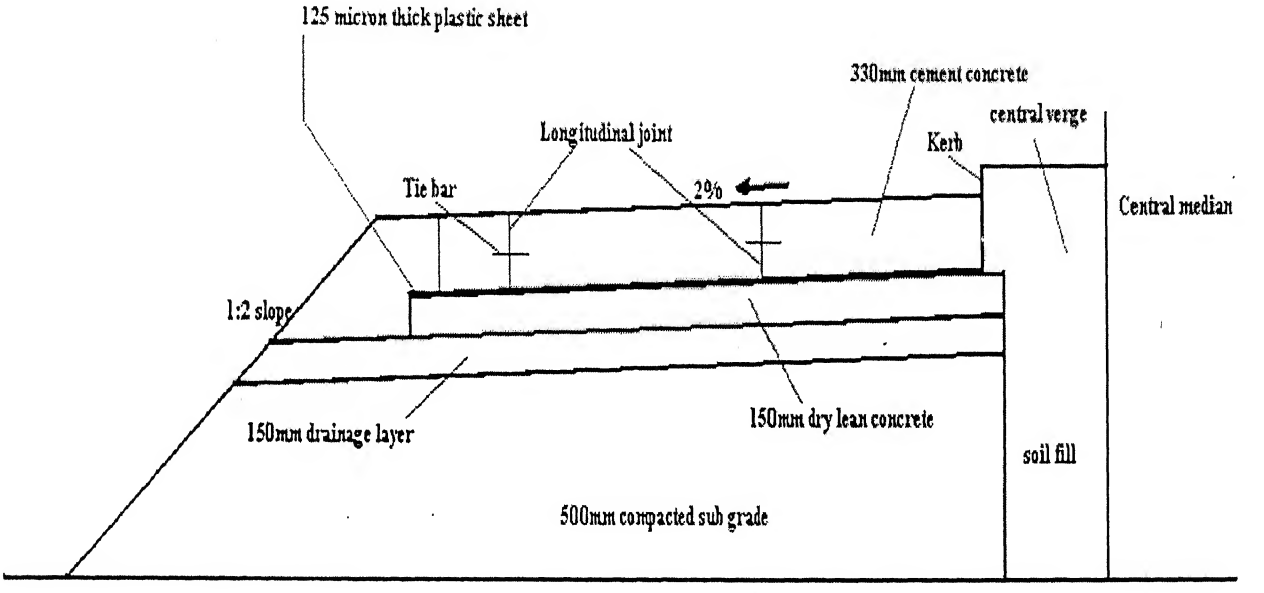


Figure A1: Cross section of the new rigid pavement

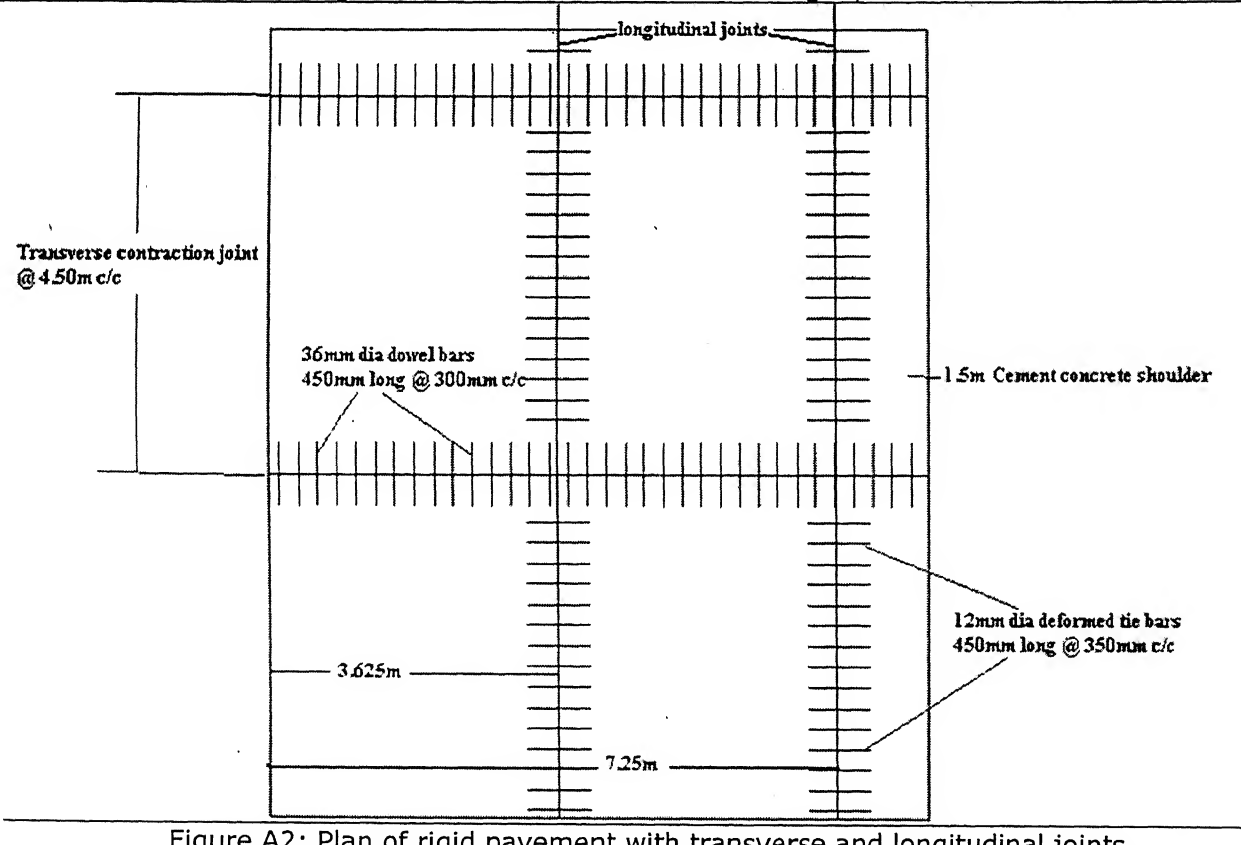


Figure A2: Plan of rigid pavement with transverse and longitudinal joints

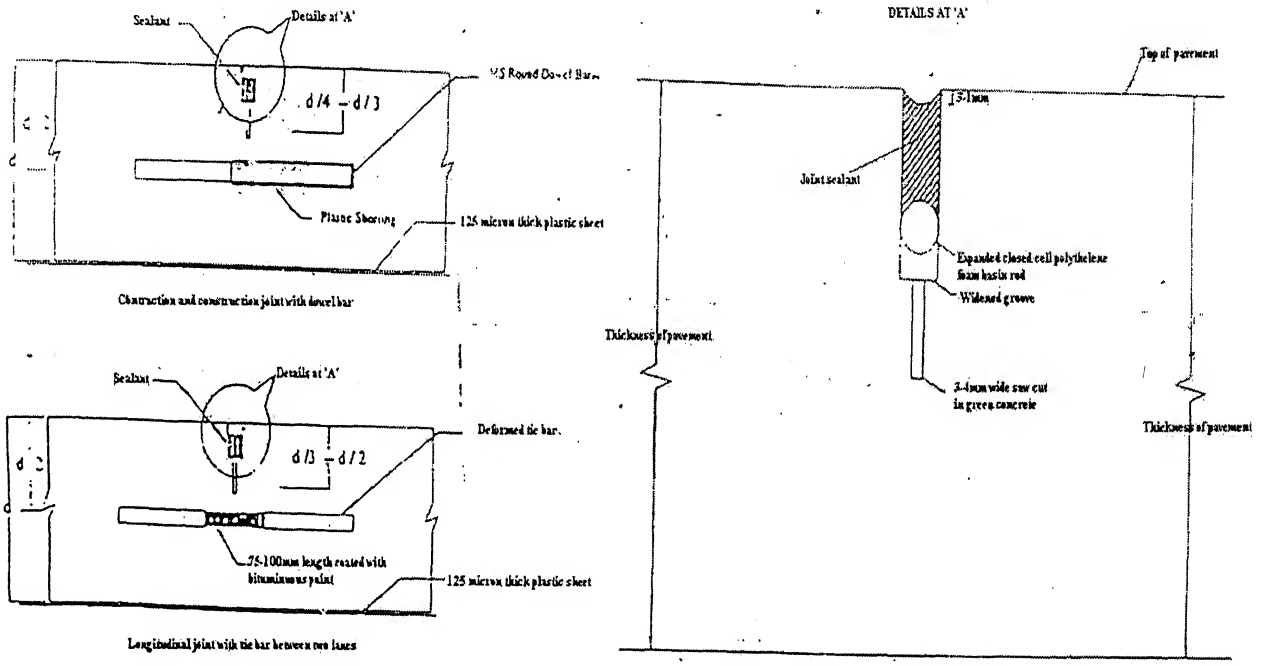


Figure A3: Details of the tie bar and dowel bar joints

Appendix B

Guidelines for Design of Plain Jointed Rigid Pavements

(IRC: 58-2002)

1. Design traffic: The cumulative traffic over the design period is given by

$$C = \frac{365 \times A\left\{(1+r)^n - 1\right\}}{r} \qquad \text{(B1)}$$

Where

C cumulative number of axles during the design period

A initial number of axles per day (3000)

r annual rate of growth of commercial traffic (7.5 per cent)

n design period in years (20 years)

$$C = \frac{365 \times 3000\left\{(1+0.075)^{20} - 1\right\}}{0.075} = 47{,}418{,}626$$

Only 25% of the traffic is to be taken, hence the design traffic will be

C = 0.25 × 47,418,626 = 11,854,657.

2. Assuming that the midpoint of the axle load class represents the group,

The expected repetitions for each axle load is given in Table B.1

Table B.1: Expected repetitions in each axle load class

Single Axle Loads		Tandem Axle Loads	
Axle load class	**Expected repetitions**	**Axle load class**	**Expected repetitions**
20	71,128	36	35,564
18	177,820	32	35,564
16	569,023	28	71,128
14	1,280,303	24	213,384
12	2,608,024	20	177,820
10	2,762,135	16	59,273
<10	3,556,397	<16	237,093

3. Taking a trail thickness of 330mm we have

As per IRC 58:2002 the relation between stress ratio and fatigue life is given as under:

N = unlimited for SR<0.45

$$N = \left[\frac{4.2577}{SR - 0.4324}\right]^{3.268} \quad \text{when } 0.45 \leq SR \leq 0.55 \qquad \text{(B2)}$$

$$Log_{10}N = \frac{0.9718 - SR}{0.0828} \quad \text{when } SR > 0.55 \qquad \text{(B3)}$$

The critical stress which is due to loading is found out and the stress ratio worked out. From the range of this ratio the allowable repetitions are found. The ratio of the expected repetition to the allowable repetition gives the fatigue life consumed. The cumulative fatigue life consumed for all axle load class should be less than one for the design to last the design life as shown in Table B.2.

Table B.2: Calculation for cumulative fatigue damage

Axle load	**AL X 1.2**	**Stress (kg/cm^2)**	**Stress ratio**	**Expected repetition (n)**	**Fatigue life (N)**	**Fatigue life consumed (n/N)**
Single Axle						
14	16.8	17.64	0.3920	1,280,303	Infinity	0
16	19.2	19.98	0.4440	569,023	Infinity	0
18	21.6	21.98	0.4884	177,820	1,403,026	0.1267
20	24.0	24.10	0.5355	71,128	190,904	0.3726
Tandem Axle						
36	43.2	19.38	0.4307	58,480	Infinity	0

The cumulative fatigue life consumed is 0.4993 which is less than one. Hence the trail thickness taken is safe.

4. The temperature stresses are obtained as per Westergaard's analysis using Bradbury's coefficient.

$$S_{te} = \frac{E\alpha tC}{2} \qquad \text{(B4)}$$

where

S_{te} temperature stress in the edge region kg/cm^2

E modulus of elasticity of concrete kg/cm^2

α coefficient of thermal expansion of cement concrete /^{o}C.

t maximum temperature differential between top and bottom of slab during day /^{o}C.

C Bradbury's coefficient which can be directly ascertained from the chart using L/l and B/l values where l is the radius of relative stiffness found from the following formula

$$l = \sqrt[4]{\frac{Eh^3}{12\left(1-\mu^2\right)k}} \text{ where} \qquad \text{(B5)}$$

h thickness of the slab

μ poisson's ratio

k modulus of sub grade reaction kg/cm^3

Substituting the values we have:

l = 103.53 cm

L/l = 450/103.53 = 4.347. The corresponding value of C is 0.5372.

The temperature differential is 21 and we get

$$S_{te} = \frac{3\times10^5\times10\times10^{-6}\times21\times0.5372}{2} = 16.9218 \text{ kg/cm}^2$$

The total edge stress is 24.10+16.9218 = 41.0218 kg/cm^2 which is less than the flexural stress of concrete (45 kg/cm^2) and hence safe.

5. The corner load stresses are obtained by

$$\text{Corner stress} = \frac{3P}{h^2}\left[1-\left(\frac{a\sqrt{2}}{l}\right)^{1.2}\right] \text{ where} \qquad \text{(B6)}$$

l relative radius of stiffness found out earlier 103.53 cm

a area of contact of the wheel, considering a single axle dual wheel

we have

$$a = \left[0.8521 \times \frac{P}{q \times \pi} + \frac{S}{\pi} \left(\frac{P}{0.5227 \times q} \right)^{0.5} \right]^{0.5} \text{ where} \qquad \text{(B7)}$$

P wheel load (8000 kg)

q tyre pressure (8 kg/cm^2)

S c/c distance between the tyres (31cm)

Substituting the values we get, a = 25.94 cm

The corner stress is 15.694 kg/cm^2 which is less than the flexural stress of concrete 45 kg/cm^2 and hence safe.

6. Design of dowel bars: The following factors are taken into account.

i. Design wheel load 8000 kg

ii. Percent load transfer 40%

iii. Joint width 2.0 cm

iv. Slab thickness 33cm

v. Dowel diameter

(assumed) 3.6 cm

vi. Permissible compressive

stresses in concrete 400 kg/cm^2

Permissible bearing stresses in concrete are calculated as under:

$$F_b = \frac{(10.16 - b) f_{ck}}{9.525} = \frac{(10.16 - 3.6)400}{9.525} = 275.49 \text{ kg/cm}^2 \qquad \text{(B8)}$$

Assuming spacing between dowel bars as 30 cm, the length of the dowel bar is 60 cm and that the first dowel bar is placed at a distance of 15 cm from the

pavement edge. Assuming that the load transferred by the first dowel is P_t, the load transferred by the dowel bar system will be given by

$$=\left[1+\frac{103.53-30}{103.53}+\frac{103.53-60}{103.53}+\frac{103.53-90}{103.53}\right]P_t = 2.2614\ P_t$$

Since the load transferred by the dowel is 40% of the wheel load we have $8000\times0.40 = 2.2614\ P_t$. Hence $P_t = 1415$kg.

<u>Check for bearing stress:</u> The moment of inertia of the dowel bar is $=\pi b^4/64 = \pi(3.6)^4/64 = 8.244\ cm^4$. The relative stiffness of the dowel bar is given by $\beta = \sqrt[4]{kb/4EI} = \sqrt[4]{\frac{41500\times3.6}{4\times2\times10^6\times8.244}} = 0.218$. Now the bearing stress in the dowel bar is found out.

$$\text{Bearing stress in dowel bar} = (P_t\times k)\times\left[(2+\beta z)/4\beta^3 EI\right] = 209.356\ kg/cm^2$$

which is less than 275.49 kg/cm^2 and hence safe.

7. <u>Design of tie bars:</u> The following parameters are taken for designing the tie bars.

i.	Slab thickness	33 cm
ii.	Lane width	3. 5m
iii.	Coefficient of friction between pavement and sub base	1.5
iv.	Density of concrete	2400 kg/cm^3
v.	Allowable bond stress in plain tie bars (B)	17.5 kg/cm^2

<u>Spacing and length of the tie bars:</u> Area of steel required per meter width of joint to resist the frictional force of the slab bottom.

$$A_s = \frac{bfW}{S} = \frac{3.5 \times 1.5 \times 0.33 \times 2400}{1250} = 3.3264 \text{ cm}^2/\text{m} \quad \text{(B9)}$$

Assuming diameter of tie bar as 16mm we have the cross sectional area as

$A = \frac{1.6^2 \times \pi}{4} = 2.01$ cm^2. Perimeter of tie bar P = $\pi \times d$ = 5.02 cm. Spacing

is = (100 × 2.01)/ 3.3264 = 60.4257 cm say, 60 cm. Length of the bar is

given by $= \frac{2 \times S \times A}{B \times P} = 57.199$ cm, say 60 cm.

Appendix C

Guide for Design of Pavement Structures, 1993 (AASHTO)

1. The AASHTO guideline is based on empirical relations which take into account the various factors such as load transfer, soil swelling, frost heave, provision of shoulder, etc,. The daily traffic is converted into Equivalent Standard Axle Loads (ESALs). The design steps are as under:

a) A trial slab thickness of 300 mm is taken. Taking the terminal serviceability as 2.5, the design traffic of 3000 commercial vehicles per day. The growth factor is 43.38. This traffic is converted into ESALs as shown in Table C.1.

Table C.1: Conversion of traffic into ESALs

Axle load (kg)	Axle Load (kips)	Traffic Equivalency Factor	Design Traffic	ESALs
Single Axle				
<10000	<22	0.080	14,250,330	1,140,026
10000	22	2.41	11,067,756	26,673,291
12000	26	5.01	10,450,252	52,355,762
14000	30	9.35	5,130,119	47,966,612
16000	36	20.4	2,280,052	46,513,060
18000	40	31.6	712,516	22,515,505
20000	45	51.3	285,006	14,620,808
Tandem Axle				
36000	80	71.9	142,503	10,245,966
32000	71	44.65	142,503	6,362,759
28000	62	25.7	285,006	7,324,654
24000	53	13.4	855,019	11,457,255
20000	45	6.645	712,516	4,734,669
16000	36	2.52	237,505	598,513
<16000	<36	0.131	950,022	124,453
			Total ESALs	252,633,333

Thus the future traffic in ESALs for the design period of 20 years is 252.6333 million standard axles. The design traffic for a two-lane highway with two way traffic is 252.633×0.5 = 126.32 million ESALs

b) Reliability: Since the project is a national highway, the reliability is set at that of the interstate highways that is 95% and the standard deviation i.e., the performance prediction error of data developed at the AASHTO test was 0.25 for rigid pavement and the same deviation is taken in this case also.

c) Design serviceability loss: The initial serviceability of the road is taken as 4.5 and the terminal serviceability for a highway is taken to be 2.5. Hence the design serviceability is 2.0.

d) Load transfer coefficient: For concrete pavements the value is between 2.5 to 3.2. For a pavement which is provided with a dowelled joint and tied PCC shoulders the value is 2.5.

e) Drainage coefficient: Considering that the pavement is exposed to moisture levels approaching saturation between 5% and 25% of the time and that the drainage is good i.e., the water drains off in a day. The value of C_d is taken as 1.1.

f) The concrete elastic modulus is taken as 3.0×10^5 kg/cm^2 which is 4.296×10^6 psi.

g) The concrete modulus of rupture $S_c^{'}$ is given as 45 kg/cm^2 which is 644.52 psi.

h) The modulus of sub grade reaction is given as 8 kg/cm^3 which is 291 pci.

2. The above values are used to interpolate the design thickness of the pavement from a nomograph which is shown in the Figure C.1a and C.1b. The thickness got was 12 inches (300 mm). In this design the effect of swelling is not considered as the data was not available and the same is not being considered for other guideline analysis. Since the data given is for Karnataka the frost heave case need not be considered.

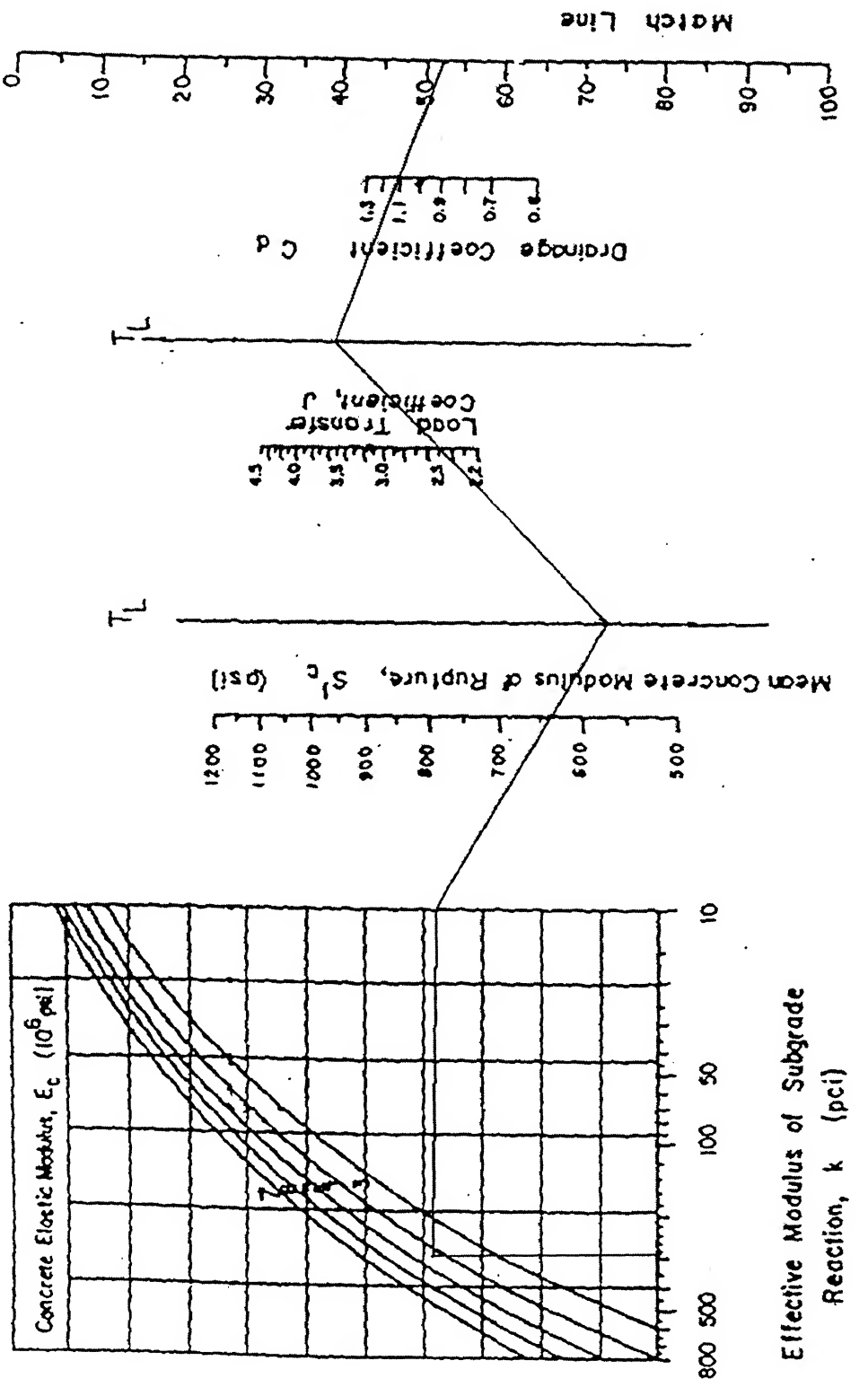


Figure C.1a: Part of the nomograph relating the various factors to the pavement thickness

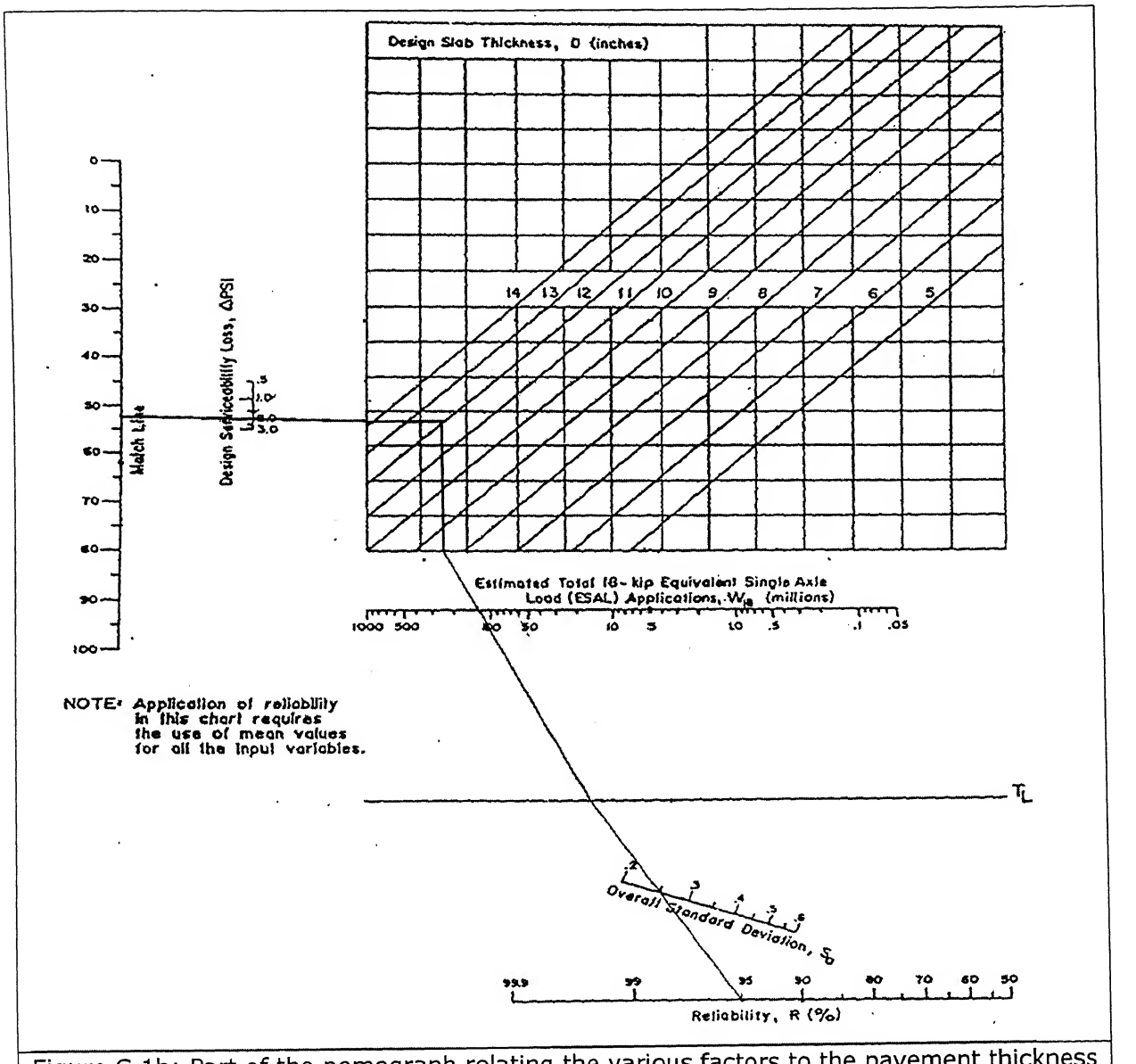


Figure C.1b: Part of the nomograph relating the various factors to the pavement thickness

Appendix D

Guide for Pavement Design (AUSTROADS 2004)

1. The project involves the provision of a two lane concrete pavement with concrete shoulders on either side. The design of the concrete pavement involves the following steps.

a) Design traffic for 20 years design period is to be found out. The equation to derive the design traffic is given by:

$$N_{DT} = 365 \times (AADT \times DF) \times \%HV / 100 \times N_{HVAG} \times LDF \times CGF \qquad \text{(D1)}$$

where

AADT Average annual daily traffic in vehicles per day which is 3000 commercial vehicles per day.

DF Direction factor which is 0.5 as the AADT given is two way.

HV As per the AUSTROADS the heavy vehicles is the vehicles of class 3 or above classification. Class 3 classification is for two axle trucks and buses. Taking that these are of load classification greater than 9 tonnes we have 70% of the vehicles of class 3 or above.

N_{HVAG} Gives the axle group per heavy vehicle and is dependent on the survey carried out. Since the data is not available for Indian conditions, presumptive numbers arrived at from data obtained at sites around Australia is used and this value for urban roads is 2.5.

LDF Since the highway is a two lane highway, the lane distribution factor is 1.0

CGF The cumulative growth factor is dependent on the design period (P) and the annual growth rate (R) and is given by

$$CGF = \frac{(1+0.01R)^P - 1}{0.01R} = \frac{(1+0.075)^{20} - 1}{0.075} = 43.31 \qquad \text{(D2)}$$

$N_{DT} = 365 \times (3000 \times 0.5) \times 70/100 \times 2.5 \times 1.0 \times 43.31 =$ 41,496,394 which is the design traffic.

b) Since the design traffic is greater than 1×10^7 a layer of 150 mm of lean mix concrete (LMC) is given. Since the strengthened sub grade modulus is given as 8 kg/cm^2, the corresponding CBR value from IRC 58:2002 is 24.62%. The project reliability considered for the project is 95% and the load safety factor (LSF) associated with this reliability for plain concrete pavement is 1.3.

c) From the above values of strengthened CBR, design traffic and LSF, the minimum base thickness is got from the design chart and is 250mm.

d) The expected repetitions calculated are shown in Table D.1. The percent of axle loads was used from the percentage given in the problem while the proportion of axle group data was not available. Hence the same available from the table giving the representative distribution of loads on axle groups for use on urban roads was used.

e) Based on the minimum base thickness and other inputs the fatigue and erosion analysis is carried out. It was found that for the thickness of 250mm taken was unsafe. Hence a higher thickness was taken and calculation carried out till fatigue and erosion analysis gave a value less than 100 when it is safe. The thickness of 290 mm was found safe in this case and the calculations are tabulated in Table D.2.

Table D.1: Expected repetition of each axle group

Axle group load (KN)	Proportion of load (%/100)	Proportion of axle group	Design HVAG	Expected repetitions
SAST (Single Axle Single Tyre)				
100	0.233	0.3799	41,496,394	3,673,124
120	0.220	0.3799	41,496,394	3,468,186
SADT (Single Axle Dual Tyre)				
140	0.108	0.2171	41,496,394	972,958
160	0.048	0.2171	41,496,394	432,426
180	0.015	0.2171	41,496,394	135,133
200	0.006	0.2171	41,496,394	54,053
TAST (Tandem Axle Single Tyre)				
160	0.025	0.0216	41,496,394	22,408
TADT (Tandem Axle Dual Tyre)				
200	0.015	0.2591	41,496,394	161,276
240	0.018	0.2591	41,496,394	193,531
280	0.006	0.2591	41,496,394	64,510
320	0.003	0.2591	41,496,394	32,255
360	0.003	0.2591	41,496,394	32,255

Table D.2: Calculation of cumulative fatigue and erosion damage

Axle group load (KN)	Expected repetitions	Stress ratio (S_r)	Fatigue analysis		Erosion analysis	
			Allowable repetition	Fatigue (in %)	Allowable repetition	Erosion (in %)
SAST (Single Axle Single Tyre)						
100	3,673,124	0.347	Unlimited	0	3.6×10^{14}	0
120	3,468,186	0.412	Unlimited	0	3.6×10^{14}	0
SADT (Single Axle Dual Tyre)						
140	972,958	0.404	Unlimited	0	5.04×10^{7}	1.93
160	432,426	0.459	1.72×10^{7}	2.51	5.05×10^{6}	8.56
180	135,133	0.512	443,000	30.50	1.58×10^{6}	8.55
200	54,053	0.566	83,000	65.12	6.88×10^{5}	7.86
TAST (Tandem Axle Single Tyre)						
160	22,408	0.281	Unlimited	0	1.01×10^{6}	2.22
TADT (Tandem Axle Dual Tyre)						
200	161,276	0.258	Unlimited	0	3.6×10^{14}	0
240	193,531	0.307	Unlimited	0	3.19×10^{7}	0.606
280	64,510	0.355	Unlimited	0	3.32×10^{6}	1.943
320	32,255	0.402	Unlimited	0	1.01×10^{6}	3.19
360	32,255	0.449	Unlimited	0	4.30×10^{5}	7.50
	Cumulative value		**fatigue**	**98.13**	**erosion**	**42.359**

2. Thus for a thickness of 290 mm, it is found that the cumulative fatigue got is 98.13% and cumulative erosion got is 42.359%, both of which are less than 100%. Hence the design is safe.

Appendix E

Design manual for Roads and Bridges, Volume 7

(TRRL Report 1132)

1. The design is standardized and the depicted by charts. The design period taken is 40 years. The daily traffic of 3000 commercial vehicles is converted to ESALs which is 150million standard axles as shown in figure E.1. Here the one way traffic for a single carriageway is taken. The equations are empirical relations based on studies of performance of full scale experimental roads and for a plain concrete pavement is given by.

$$Ln(L) = 5.094Ln(H) + 3.466Ln(S) + 0.4836Ln(M) + 0.08718Ln(F) - 40.78 \qquad \text{(E1)}$$

where

ln is the natural logarithm

L is the cumulative traffic in msa

H is the slab thickness in mm

S is the 28 day mean compressive strength in MPa

M is the equivalent foundation modulus in MPa

F is the percentage failed bays

2. Thus based on the design traffic the thickness can be calculated. The design traffic in ESALs for a traffic density of 3000 commercial vehicles per day is 150 million standard axles (msa), which is for a design period of 40 years. The thickness of the pavement (Equation 4 above) in millimeters can be obtained by substituting as follows: L (msa) = 150, S (compressive strength at 28 days in MPa) = 45, M (foundation modulus for sub grade of 2% and a sub base of 150mm dry lean concrete given in MPa) = 270 and F (given as % of bays failed and taken as 30% for unreinforced concrete = 30 which was

333.4mm. This thickness is for a pavement without shoulders. On the provision of concrete shoulders the thickness is reduced by the magnitude given in Figure E.2 which is 34mm for a thickness of 330mm. Thus the reduced thickness for a pavement with tied shoulders is 299.4mm ≈ 300mm.

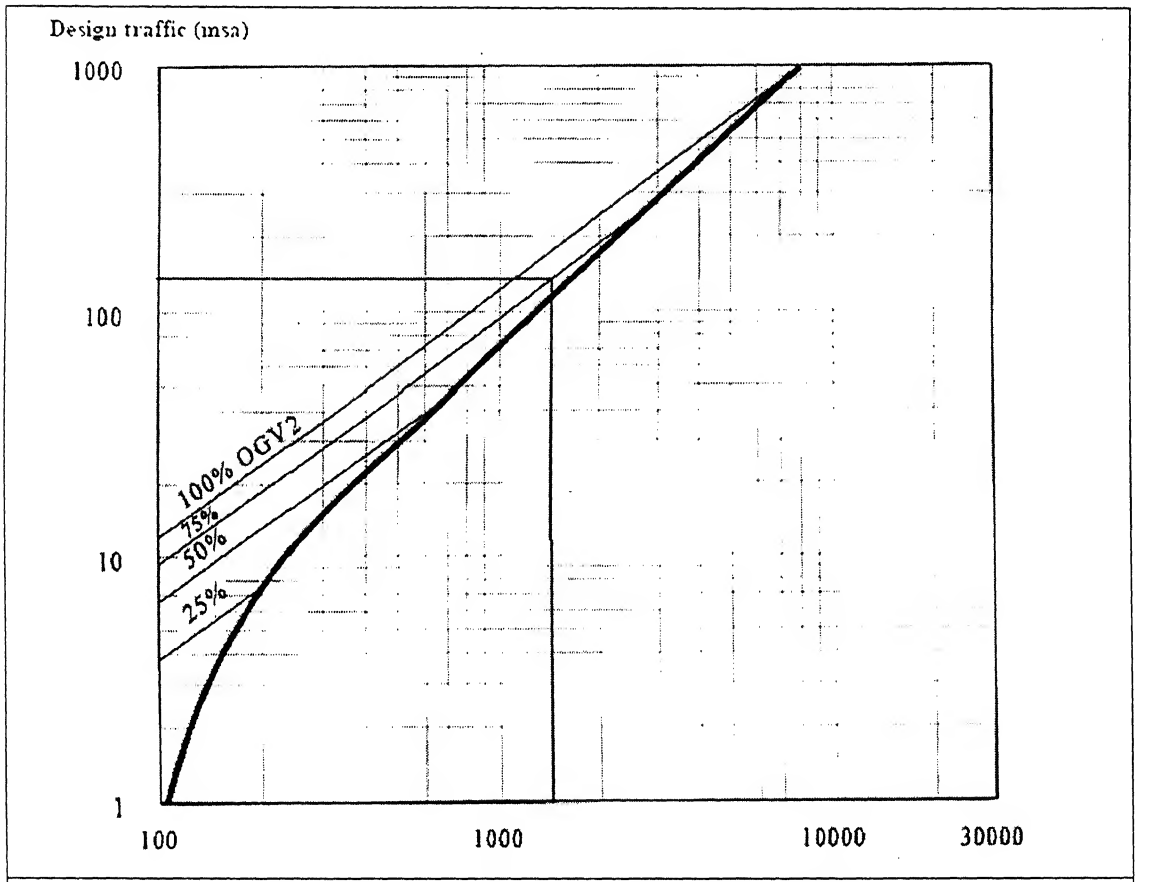
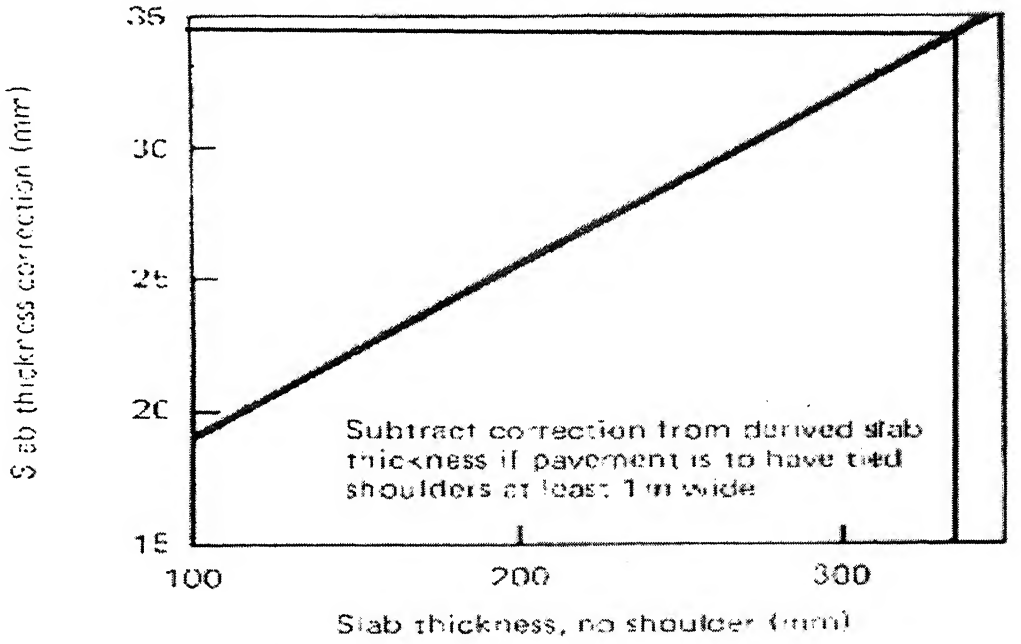


Figure E.1: Conversion of daily traffic to ESALs



Contribution of tied shoulder to pavement life in terms of equivalent slab thickness

FigureE.2: Graph relating tied shoulder to equivalent slab thickness